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Properties of Concrete made with North Carolina Recycled Coarse and Fine Aggregates

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16. Abstract

Reconstruction of roadways generates large quantities of waste material and requires considerable amounts of aggregate. The recycling of concrete from old deteriorated pavements into aggregates for construction of new pavements reduces disposal costs as well as providing a source of aggregates to replace natural supplies.

In this study, recycled coarse and fine aggregates were obtained from a portion of concrete pavement which was removed from Interstate 40 in North Carolina. Various amounts of recycled coarse and fine aggregates were volumetrically substituted for natural coarse and fine aggregates of a control mixture. A relatively higher cement factor was used for the control mix, compared to NCDOT standard. The effects on plastic and hardened concrete properties were investigated. Compared to natural aggregates, the recycled aggregates had lower specific gravities and higher water absorptions. Compared to concrete made with natural aggregate, recycled aggregate concretes had less slump and air content, but could be controlled with increasing the dosage of chemical admixtures. Unit weight of the recycled aggregate concretes were lesser than that for concrete with natural aggregates. All strength characteristics, including compressive strength, elastic modulus, flexural strength, and splitting tensile strength, decreased with increasing percentage of recycled fine aggregate. The empirical relationships between the strength properties were investigated and the results were compared to predictions using available empirical equations in the specifications.

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PROPERTIES OF CONCRETE MADE WITH NORTH CAROLINA RECYCLED COARSE AND FINE AGGREGATES

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EXECUTIVE SUMMARY

For both environmental and economic reasons, there is a significant need for the reuse of demolished concrete pavements. The process of crushing concrete produces two types of material: recycled coarse aggregate and recycled fine aggregate. The use of recycled coarse aggregate has been well established, and this project focused on the possible use of the recycled fine aggregate in concrete. The effects on the plastic and hardened properties of recycled aggregate concrete consisting of 60% or 100% recycled coarse aggregate (RCA) and 0%, 25%, 50%, 75%, or 100% recycled fine aggregate (RFA) were investigated.

The objectives of the study were:

(i) *To determine the physical properties of North Carolina recycled aggregates. This included:*

- *Specific Gravity*
- *Dry Rodded Unit Weight*
- *Water Absorption*
- *Sieve Analysis*
- *Fineness Modulus*

(ii) *To determine if it is possible to produce concrete using North Carolina recycled aggregates, both in the coarse and fine fractions. Investigate the effects of these aggregates on the fresh concrete properties, including:*

- *Slump*
- *Air Content*
- *Unit Weight*

(iii) *To evaluate the effects of the replacement of natural aggregates by recycled aggregate on the mechanical properties of hardened concrete. This included:*

- *Compressive Strength*
- *Elastic Modulus*

- *Flexural Strength*
- *Splitting Tensile Strength*

(iv) *To investigate the validity of AASHTO and ACI empirical relationships for estimating the elastic modulus, flexural strength, and splitting tensile strength of recycled aggregate concretes.*

(v) *To study the effects of using recycled aggregates on interfacial shear bond strength, and to compare and evaluate the results from two notable interfacial bond test methods.*

Based on the results of this study, using the materials and ranges of variables considered, the following conclusions can be drawn for concretes made with recycled coarse and fine aggregates:

1. Increasing the percentage of recycled fine aggregate resulted in a loss of slump, and hence, workability. This could be adjusted by addition of a water reducer to bring the slump back within the target slump of 50 mm ± 25 mm (2 in. ± 1 in.).
2. As the percentage of recycled fine aggregate increased, the air content of the fresh concrete decreased. To meet the target air content of 5.5% ± 1.5%, higher dosages of air entraining agent were used. For the concrete mixture utilizing all recycled aggregates it was not possible to produce a mixture which satisfied the target air content requirements.
3. Increasing the percentage of recycled aggregate decreased the unit weight of concrete. This was expected since the recycled aggregates had lower specific gravities than the natural aggregates.
4. For hardened concrete, the compressive strength, elastic modulus, flexural strength, and splitting tensile strength showed no clear trend with the substitution of recycled coarse aggregate in the range considered in this study. These properties, however, did show an overall decrease with the substitution of recycled fine aggregate. The relative magnitude of this decrease was independent of the amount of recycled coarse aggregate used in the mixture.

5. The compressive strength at the test age of 28 days for concretes made with 100% recycled fine aggregate was 25% to 30% lower than comparable concretes made with 100% natural fine aggregate.
6. The elastic modulus at the test age of 28 days for concretes made with 100% recycled fine aggregate was 28% to 40% lower than comparable concretes made with 100% natural fine aggregate.
7. The flexural strength at the test age of 28 days for concretes made with 100% recycled fine aggregate was 15% to 20% lower than comparable concretes made with 100% natural fine aggregate.
8. The splitting tensile strength at the test age of 28 days for concretes made with 100% recycled fine aggregate was 18% to 27% lower than comparable concretes made with 100% natural fine aggregate.
9. Based on the variability and range of the experimental data, the empirical equation recommended by AASHTO and ACI 318 for estimating elastic modulus seems to be applicable for recycled aggregate concretes.
10. The empirical relationships recommended by ACI 330 and ACI 325 for estimating the flexural strength as a function of compressive strength are unconservative for recycled aggregate concretes. The relationship recommended in ACI 318-95 represented a lower bound estimate of the experimental data.
11. The ACI 318-95 equation for splitting tensile strength is unconservative for recycled aggregate concretes. For the experimental data, the lower bound was represented by $f_{ct} = 0.54\ f_c$ for SI units and $f_{ct} = 6.5\ f_c$ for US Customary units. The AASHTO recommendation for estimating the splitting tensile strength as 86% of the flexural strength was found to unconservative for recycled aggregate concretes. The experimental data showed that for concretes with recycled aggregates, the average splitting tensile strength is 78% of the flexural strength.
12. The Double L Interfacial Bond Strength Test is a reliable test for obtaining interfacial bond strength values. Results from these tests were very consistent for replicate specimens. For concretes with recycled aggregate, interfacial bond strength decreases nonlinearly with increasing volume percentage of recycled fine aggregate.

13. Interfacial bond strength of recycled aggregate concretes (60% RCA and 0% RFA 50%) to natural aggregate concrete was higher than that for NCDOT mixtures using natural aggregates. This can be attributed to the increased cement content used for concrete mixtures with recycled aggregates.
14. Interfacial bond strengths computed from the Double L Interfacial Bond Strength Test were higher than the estimated tensile strength of the weaker concrete (i.e. concrete with recycled aggregates). This shows that adequate interfacial bond strength was developed and failure in this type of overlay pavement will most likely be governed by the tensile strength of the recycled aggregate concrete.
15. For the ASTM C882-91 Slant Shear Test, over three quarters of the specimens did not fail in the intended mode when tested at the design age of 7 days for the overlay. If specimens had failed along the inclined bond plane, the computed bond strength would have been 2.5 times greater than that conputed from the Double L interfacial bond test.

Based on the results of this study, it can be determined that recycled coarse aggregate can be used as a substitute for natural coarse aggregates. However, recycled fine aggregate is not acceptable as a complete substitute for natural fine aggregate. It may, however, be suitable as a partial replacement for natural fine aggregate. Most departments of transportation specify either no recycled fine aggregate, or less than 30% replacement of natural sands with recycled fines. A similar specification that is often used is to allow up to 25% replacement of natural sands with manufactured sands. Manufactured sands, like recycled fine aggregates, are very harsh and difficult to finish.

The North Carolina Department of Transportation (NCDOT) requirement of a minimum flexural strength of 3.79 MPa (550 psi) at 14 days was met by the mixtures with 60% recycled coarse aggregate mixtures, as long as the volumetric percentage of recycled fine aggregate was less than 50 percent. Considering the field conditions and imposing a margin of safety, it is recommended that concrete mixtures with 60% RCA and up to 30% RFA can be used for pavement applications.

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1. INTRODUCTION

1.1 Background

In 1975, a joint research project by the Texas State Department of Highways and the Texas Transportation Institute included a survey on litter disposal and the use of waste materials [1]. Part of the results indicated that:

"Very few states were giving consideration to the reuse of existing road-bed materials for rehabilitation and reconstruction uses other than for unstabilized base courses. The normal disposal of asphalt concrete or portland cement concretes seem to have been in landfills or for riprap in drainage ditches. While the riprap idea has merit, the disposal of these materials in landfill areas is particularly questionable today, due to both the need to conserve our valuable resources and the relatively high cost of providing new construction materials."

Much has changed in the 20 years since that report was issued. By the end of 1989 eight states - Iowa, Minnesota, Illinois, Michigan, Wisconsin, North Dakota, Wyoming, and Oklahoma - had all been engaged in projects that recycled portland cement concretes into new pavements [2]. Today, the recycling of pavement concretes into new pavements is almost automatically considered as an option when it comes time to repair or reconstruct a roadway. The American Concrete Pavement Association recently published a document containing recommendations for selecting pavement rehabilitation strategies [3]. Included as an option was the reconstruction of existing roadways by recycling the pavement as aggregate.

The increase in the recycling of concrete has come about for several reasons. The first of these is the increase in landfill costs associated with dumping waste materials. The reduction in availability of good, high-quality natural aggregates is a second reason. In addition, as aggregates become more scarce and landfills close down, the increase in transportation costs both for disposal of old material and procurement of new materials has risen dramatically. These transportation costs are especially significant in urban areas, which are typically far removed from aggregate sources and landfills. In some projects, virgin aggregates have been hauled 322 km (200 miles) for use [4].

Another important factor concerning the increase in use of recycled pavements is that much of the preliminary work has already been performed in various field studies.

The first recycled concrete projects had to overcome several difficulties. Among them was separation of any asphalt overlays, breaking of the concrete slab, removal of any reinforcing steel, and final crushing of the concrete. As the various projects progressed, contractors approached these difficulties in different ways. Today, these various techniques are available to contractors and agencies starting their own recycled concrete projects. The American Concrete Pavement Association publishes a guide which includes the different techniques available [4]. With this type of information available, the use of recycled aggregate concrete can be expected to become more widespread as the economic advantages become more pronounced.

1.2 Objectives

The objectives of this investigation were:

(i) *To determine the physical properties of North Carolina recycled aggregates. This included:*

- *Specific Gravity*
- *Dry Rodded Unit Weight*
- *Water Absorption*
- *Sieve Analysis*
- *Fineness Modulus*

(ii) *To determine if it is possible to produce concrete using North Carolina recycled aggregates, both in the coarse and fine fractions. Investigate the effects of these aggregates on the fresh concrete properties, including:*

- *Slump*
- *Air Content*
- *Unit Weight*

(iii) *To evaluate the effects of the replacement of natural aggregates by recycled aggregate on the mechanical properties of hardened concrete. This included:*

- *Compressive Sytength*
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(iv) *To investigate the validity of AASHTO and ACI empirical relationships for estimating the elastic modulus, flexural strength, and splitting tensile strength of recycled aggregate concretes.*

(v) *To study the effects of using recycled aggregates on interfacial shear bond strength, and to compare and evaluate the results from two notable interfacial bond test methods.*

1.3 Scope

The basic premise of the testing program was to use a "control" concrete mixture and determine the effect of replacing the natural aggregates with recycled aggregates. The natural coarse aggregate was replaced volumetrically by either 60% or 100% recycled coarse aggregate. The natural fine aggregate was replaced volumetrically by either 0%, 25%, 50%, 75%, or 100% recycled fine aggregate. The effects on both fresh and hardened concrete properties were determined and evaluated. The validity of AASHTO and ACI empirical relationships to estimate elastic modulus, flexural strength, and splitting tensile strength for recycled aggregate concrete was investigated.

2. LABORATORY INVESTIGATION

This chapter discusses the experimental investigation, including the materials and procedures used in specimen fabrication and testing, and the tests that were performed.

First, the materials used in the production of the specimens will be discussed. Second, the mixture proportions and the procedures used in specimen fabrication will be reviewed. Third, the equipment used in testing and the tests that were conducted will be addressed.

The experimental investigation included tests on fresh and hardened concrete. For the fresh concrete, the tests performed were slump, air content, and unit weight. For the hardened concrete, the tests performed were compressive strength, elastic modulus, flexural strength, and splitting tensile strength. For each of the hardened concrete tests, a number of replicate specimens were tested at various ages and for different percentage replacement of natural coarse and fine aggregates with recycled coarse and fine aggregates. The test ages were 14 days, 28 days, and 9 months. Each batch series consisted of either 60% or 100% replacement of natural coarse aggregate with recycled coarse aggregate, and 0%, 25%, 50%, 75%, and 100% replacement of natural fine aggregate with recycled fine aggregate. Two "control" mixtures with no recycled aggregates was also produced and tested. The first "control" mixture termed as NCDOT mix used 562 pcy (pounds per cubic yard) of cement and the second "control" mixture termed as NCSU mix used 620 pcy of cement. The higher cement factor for the NCSU mix was used in anticipation of the strength degradation due to the use of recycled aggregates as replacement of natural and fine aggregates. A summary of the test program for the laboratory investigation is shown in **Table 2.1**.

2.1 Constituent Materials

2.1.1 Cement

Type I Cement was used for all phases of the testing program. The cement was manufactured by Giant Cement, located in Harleyville, South Carolina. Although the cement was ordered in three different batches, each batch met the requirements of ASTM C150, *Specification for Portland Cement* [5].

2.1.2 Natural Coarse Aggregate

All of the natural coarse aggregate used in this program was a #57 (25 mm or 1 inch nominal maximum size) crushed granite aggregate. Garner Quarry in North Carolina was the source of the coarse aggregate. A physical analysis of the coarse aggregate was performed according to ASTM C33, *Specification for Concrete Aggregates* [5]. Results of the analysis are shown in **Table 2.2**. The gradation of the coarse aggregate satisfies ASTM requirements for minimum and maximum percent passing limits (**Table 2.2**).

2.1.3 Natural Fine Aggregate

The natural fine aggregate was a siliceous sand from Lillington, North Carolina. Physical analysis of the sand was performed according to ASTM C33, *Specification for Concrete Aggregates* [5]. The results from this analysis are shown in **Table 2.3**. Also shown in **Table 2.3** are gradation values for the fine aggregate which satisfies the ASTM gradation requirements.

2.1.4 Recycled Aggregates

The recycled aggregates were obtained from a concrete slab identified by the Pavement Management Unit of the North Carolina Department of Transportation (NCDOT). The slab was obtained from Davie County and the material was removed from Interstate 40, Milepost 165, Station 675+00, in the east-bound lane. This location is close to the US 64 exit at the Iredell-Davie County line near Hunting Creek. Information from the NCDOT indicates that Smithgrove, North Carolina was the source of the course aggregate. The fine aggregate which was used in the original concrete mixture was provided by Yadkin River natural sand company. The roadway was originally opened in December of 1970. The pavement was removed due to excessive transverse cracks which had exposed the reinforcing bars.

Once the slab was identified, a small section was transported to the NCDOT Materials and Tests Unit for testing. A photograph of the test slab is shown in **Figure 2.1**. Two compression tests and two split cylinder tests were performed on cylinders cored from the slab. These cores were approximately 81 x 146 mm (3.2 x 5.75 inches). Results from these tests, performed by NCDOT Materials and Tests Unit, are shown in **Table 2.4**. The results of the compressive tests were adjusted to account for a diameter to height ratio of less than 2:1.

Additionally, the Materials and Tests Unit performed acid soluble chloride content tests at varying depths on the concrete slab. The results of these tests are shown in **Table 2.5**. The first set of tests show a decreasing amount of chloride ion with increasing depth. The second set, however, shows no clear trend. This variability may be due to material differences in the sample cored for testing. The presence of chloride ion in recycled aggregates has been a cause for concern due to the increased risk of steel corrosion. However, with an average chloride content of 1.50 kg/m^3 (0.89 lb/yd^3), this material is well below the chloride content of recycled aggregates used in other projects [2].

The remainder of the concrete slab was crushed by Phoenix Recycling Corporation located in Havelock region, North Carolina. It was intended that this would approximate a field crushing operation as closely as possible. The crushed aggregate was received at NCSU in November, 1994. The unsieved material is shown in **Figure 2.2**. The crusher product was sieved to obtain coarse and fine fractions. The coarse sizes were stockpiled separately, then recombined to form a #57 (25 mm or 1 inch nominal maximum size) aggregate. The recycled fine aggregate was considered to be any material passing the 2.36 mm (#8) sieve. Tests on the recycled coarse and fine aggregates were performed according to ASTM C33, *Specification for Concrete Aggregates* [5]. Results for the recycled coarse aggregate are shown in **Table 2.6**. Results for the recycled fine aggregate are shown in **Table 2.7**. The recycled coarse aggregate was recombined to ensure that it met the specification for an ASTM #57 aggregate. To replicate field conditions, the recycled coarse aggregate was not washed prior to use. With more than 10% passing the 150 μm (#100) sieve, the recycled fine aggregate did not meet the requirements in ASTM C33. Further sieving revealed that approximately 6% of the recycled fine aggregate passed the 75 μm (#200) sieve. However, one of the objectives of the study was to use the recycled fine aggregate as produced by the crushing operation.

Therefore, no modifications were made to the recycled fine aggregate. A comparison of the grain size distribution for the natural and recycled aggregates is shown in **Figure 2.3**. A pictorial comparison of the size and texture of the recycled and natural aggregates is shown in **Figure 2.4**.

2.1.5 Chemical Admixtures

Two types of chemical admixtures were used during this investigation, an air entraining agent (AEA), and a high range water reducer (HRWR). The AEA used was a neutralized vinsol resin with the brand name of Daravair, manufactured by W.R. Grace and satisfying ASTM C260, *Standard Specification for Air-Entraining Admixtures for Concrete* [5]. The HRWR used was a sulfonated naphthalene type by the brand name of PSI-Super, also manufactured by W.R. Grace and satisfying ASTM C494, *Standard Specification for Chemical Admixtures for Concrete*, Type F [5].

2.2 Mixture Proportions

2.2.1 Control Mixtures

To investigate the effects of recycled aggregates, two control mixtures were developed. The first of these was based on a design mixture provided by the NCDOT from APAC-Georgia, Inc. It is a concrete pavement mixture, with cement factor of 312 kg/m^3 (526 pcy) and a minimum flexural strength of 3.79 MPa (550 psi) at 14 days. The water-cementitious (w/c) ratio of this mixture was 0.47, and this ratio was used in all subsequent mixtures. This mixture was reproduced at NCSU using the available materials to verify that the required flexural strength could be achieved with these materials. The mixture proportions for this mixture, designated "NCDOT Mix",are shown in **Table 2.8**.

A second control mixture was then developed using a cement factor of 368 kg/m^3 (620 pcy). This mixture, designated "NCSU Mix 1", used the same ratio of coarse to fine aggregate as the "NCDOT Mix". This mixture design formed the basis of all subsequent mixtures using recycled aggregates, and had a higher cement factor to partially offset the drop in strength anticipated upon substitution of natural aggregates with the recycled aggregates. The mixture proportions for "NCSU Mix 1" are shown in **Table 2.8**.

2.2.2 Mixtures Utilizing Recycled Aggregates

Once the mixture proportions for the control mixture "NCSU Mix 1" were established, two series of mixtures with recycled aggregates were cast. Each series had a designated amount of recycled coarse aggregate (RCA), either 60% or 100%, substituted volumetrically for the natural coarse aggregate. Within each series, the substitution of natural fine aggregate with recycled fine aggregate (RFA) was performed. Each batch in the series had either 0%, 25%, 50%, 75%, or 100% of the natural fines replaced by recycled fines. Increasing the amount of recycled material in the mixture required additional amounts of AEA to maintain the target air content of 5.5% ± 1.5%, and additional HRWR to maintain a target slump of 50 mm ± 25 mm (2 in. ± 1 in.). During the course of mixing, the w/c ratio was kept constant at 0.47 for all mixtures. The mixture proportions for all batches, including the "NCDOT Mix" and "NCSU Mix 1" are detailed in **Appendix A.**

To keep the w/c ratio constant, the amount of free water was adjusted to account for the water content of the aggregates. Water was added or removed from the mixture based on the saturated, surface dry absorption of the aggregate. However, according to Hansen [6], the procedure used to determine the water absorption of fine aggregates is not suitable for use with recycled fine aggregates. Since strength is primarily based on the water-cement ratio, an inaccurate absorption value will result in an unknown w/c ratio. Puckman and Henrichsen [7] have suggested a new procedure based on a drying curve, but this has yet to be standardized.

2.3 Specimen Preparation

2.3.1 Mixing Procedure

A mixing procedure was developed in the early stages of the research. This mixing procedure was developed to ensure that the concrete was adequately mixed. The specimens were all prepared at the NCSU Concrete Materials Laboratory using a tilt drum mixer with a capacity of 0.099 m (3.5 ft^3). The mixing procedure for all mixes is summarized as follows:

1. One day before mixing, coarse and fine aggregates were weighed and covered up to prevent either absorption or evaporation of moisture from the air. At this time, representative samples of the aggregates were taken for determination of water content.
2. On the day of mixing, the water content of the aggregates was determined. The weight of the aggregates and the amount of mixing water was then adjusted to account for the water content of the aggregates.
3. The concrete mixer was buttered with a mortar consisting of Type I cement and a representative sample of the fine aggregates for the mixture. The mixer was then turned upside down for 1 minute to allow excess water and mortar to drain from the mixer.
4. Three-quarters of the coarse and fine aggregates was added to the mixer along with about one-half of the mixing water and all of the AEA. This was mixed for 2-3 minutes until a uniform and well distributed mixture was obtained.
5. All of the cement, the remaining coarse and fine aggregates, and most of the remaining water was then added to the mixer. About one-half to three-quarters of the HRWR was also added. This mixture was then allowed to mix for 2-3 minutes.
6. To achieve the target slump of 50 mm ± 25 mm (2 in. ± 1 in.), the remaining water and small amounts of HRWR were then added to the mixer. Once the target slump appeared to have been achieved, the concrete was allowed to mix for 4-6 minutes. Total mixing time for each batch was approximately 8-10 minutes. Once the mixture was ready, fresh concrete testing and specimen fabrication was performed. The final amounts of water and HRWR used was recorded on the batch sheet.

2.3.2 Specimen Fabrication and Curing

All specimens prepared and used in this program were prepared and cured in accordance with ASTM C192, *Practice for Making and Curing Concrete Test Specimens in the Laboratory* [5]. The specimens used for the compression, elastic modulus, and splitting tension tests were 100 x 200 mm (4 x 8 inch) cylinders, which were cast using metal molds. The beams used in the flexural test were 100 x 100 x 356 mm (4 x 4 x 14 inch) specimens, and were cast using plastic (PVC) molds. The cylinders were consolidated by hand rodding, and the beams were consolidated by a rotating vibrator. **Figure 2.5** shows a typical setup for casting of specimens.

2.4 Testing Equipment

2.4.1 Loading Equipment

All of the tests for this study were conducted at the NCSU Structures and Materials Laboratory. The testing of the compressive strength, elastic modulus, flexural strength, and splitting tensile strength was conducted using a Baldwin Universal Testing Machine with a capacity of 535 kN (120 kips). This machine was equipped with a M120BTE Automated Control System manufactured by SATEC Systems, Inc.

2.4.2 Deformation Measurement

Deformations were measured during the compression testing to obtain values for the modulus of elasticity. Two linear voltage differential transducers (LVDTs) were attached to the specimens using an aluminum jig to obtain a constant gage length of 100 mm (4 inches). The LVDTs used were Trans-Tek, Inc. Gaging Transducers DC-DC Series 350-0000 with a sensitivity of 4.500 V/V/25.4 μm (4.500 mV/V/ 0.001 inch). The LVDTs were attached to the concrete specimens using a cyanoacrylic gel. Output from the LVDTs was digitized by an AT-MIO-16 12 bit analog to digital converter manufactured by National Instruments. The load and displacement outputs were recorded by LabWindows data acquisition software on an IBM compatible 386 computer.

2.5 Testing

2.5.1 Fresh Concrete Testing

The testing of fresh concrete properties, and the results from these tests are discussed below. For each batch in the study, the data from the testing of fresh concrete properties is presented in **Appendix B**.

2.5.1.1 Slump

One of the goals for this investigation was to achieve a workable mixture, and thus ensure adequate field placement. Since this concrete was designated as a pavement concrete, a target slump of 50 mm ± 25 mm (2" ± 1") was prescribed for each mixture. The measurement of slump was performed according to ASTM C143-90, *Test Method for Slump of Hydraulic Cement Concrete* [5].

2.5.1.2 Air Content

On the basis of the NCDOT's design specification for freeze-thaw durability, a target air content of 5.5% ± 1.5% was specified. Testing of the air content was performed according to ASTM C231-89, *Test Method of Air Content of Freshly Mixed Concrete by the Pressure Method* [5]. A Press-Ur-Meter manufactured by Charles R. Watts Company, meeting the requirements of an ASTM C-231 Type B meter was used to conduct the air content tests.

2.5.1.3 Unit Weight

The unit weight of all concrete mixes was measured according to ASTM C138-81, *Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete* [5]. In general, as the amount of recycled material increased, the unit weight of the concrete decreased. This was expected since the specific gravity of the recycled aggregates was lower than that of the natural aggregates, and substitution was done on a volumetric basis.

2.5.2 Hardened Concrete Testing

Results from testing of hardened concrete specimens are presented using average values of replicate specimens. Test data of individual specimens is presented in **Appendix C.**

2.5.2.1 Elastic Modulus

The modulus of elasticity was obtained by attaching to LVDTs on opposite sides of a 100 x 200 mm (4 x 8 inch) cylinder. As the cylinder was tested in compression, the data acquisition software LabWindows recorded the voltage change of the LVDTs and of the internal load cell of the testing machine. These voltages were then converted to stress and strain values using appropriate conversion factors. The stress-strain curve was then plotted, and the elastic modulus was calculated by performing a linear regression between 50 micro-strain and 40% of the ultimate stress. The elastic modulus of elasticity was obtained at the ages of 14 days, 28 days, and 9 months. The tests for the modulus of elasticity were conducted on the same specimens which were used for the compressive strength tests. The test setup for this test is shown in **Figure 2.6**.

2.5.2.2 Compressive Strength

The compressive strength of the 100 x 200 mm (4 x 8 inch) cylinders was determined according to ASTM C39, *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens* [5]. However, instead of sulfur caps, an unbonded capping system consisting of steel restraining rings and neoprene rubber pads was used. The cylinders were tested at 14 days, 28 days, and 9 months. They were loaded to failure at a rate of 117 kN (26,400 pounds) per minute. The test setup is shown in **Figure 2.6**.

2.5.2.3 Flexural Strength

The flexural strength of the concrete specimens was determined according to ASTM C78-84, *Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)* [5]. For each beam specimen, the flexural strength was determined by loading the beam at third points over a 305 mm (12 inch) clear span. To minimize the torsional effect that is possible due to the specimens not having perfectly square bottoms, one support was manufactured so as to be free to rotate in the direction transverse to the beam's longitudinal axis.

The 100 x 100 x 356 mm (4 x 4 x 14 inch) beams were tested at the ages of 14 days, 28 days, and 9 months. Companion 100 x 200 mm (4 x 8 inch) compression cylinders were also tested as described in section 2.5.2.2. Beam specimens were loaded to failure at a rate of 3.6 kN (800 pounds) per minute. The test setup for the flexural test is shown **in Figure 2.7.**

2.5.2.4 Splitting Tensile Strength

The determination of splitting tensile strength was done according to ASTM C496-90, *Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens* [5]. Testing was performed on 100 x 200 mm (4 x 8 inch) specimens, which were loaded to failure at a rate of 33.4 kN (7500 pounds) per minute. The setup for the splitting tension test is shown in **Figure 2.8.**

Table 2.1. Summary of Test Program

Mix ID	Percent RFA	Compressive Strength			Elastic Modulus			Flexural Strength			Split Tensile Strength		
		14*	28	270	14	28	270	14	28	270	14	28	270
NCDOT Mix cement content 312 kg/m^3 (526 pcy)	0%	3**	3	3	3	3	3	3	3	3	3	3	3
NCSU Mix 1 cement content 368 kg/m^3 (620 pcy)	0%	3	3	3	3	3	3	3	3	3	3	3	3
60% RCA	0%	3	3		3	3		3	3	3	3	3	3
cement	25%	3	3	3	3	3	3	3	3	3	3	3	3
content	50%	3	3	3	3	3	3	3	3	3	3	3	3
368 kg/m^3	75%	3	3	3	3	3	3	3	3	3	3	3	3
(620 pcy)	100%	3	3	3	3	3	3	3	3	3	3	3	3
				3			3						
100% RCA	0%	3	3	-	3	3		3	3		3	3	-
cement	25%	3	-	-	3	-	-	3	-	-	3	-	-
content	50%	3	-	-	3	-	-	3	-	-	3	-	-
368 kg/m^3	75%	3	-	-	3	-	-	3	-	-	3	-	-
(620 pcy)	100%	-	-	-	-	-	-	-	-	-	-	-	-
							-			-			-

Notes:
RCA - Recycled Coarse Aggregates
RFA - Recycled Fine Aggregate
* - Test Age in Days
** - Number of Replicate Specimens

Table 2.2. Properties of Natural Coarse Aggregate

Property	Crushed Granite	ASTM Minimum	ASTM Maximum
Sp. Gr. (SSD)	2.64	-	-
% Absorption	0.57	-	-
DRUW, kg/m^3 (pcf)	1500 (93.6)	-	-
Sieve Size	**% Passing**		
38.1 mm (1 1/2")	100.0	100	100
25.4 mm (1")	100.0	95	100
19.1 mm (3/4")	87.6	-	-
12.7 mm (1/2")	25.9	25	60
4.75 mm (#4)	1.9	0	10
2.36 mm (#8)	1.3	0	5

Table 2.3. Properties of Natural Fine Aggregate

Property	Lillington Sand	ASTM Minimum	ASTM Maximum
Sp. Gr. (SSD)	2.57	-	-
% Absorption	1.1	-	-
Fineness Modulus	2.66	-	-
Sieve Size	**% Passing**		
4.75 mm (#4)	100.0	95	100
2.36 mm (#8)	91.5	80	100
1.18 mm (#16)	69.3	50	85
600 µm (#30)	37.7	25	60
300 µm (#50)	12.4	10	30
150 µm (#100)	3.1	2	10

Table 2.4. Mechanical Properties of Concrete Slab for Obtaining Recycled Aggregates

Sample #	Test Type	Strength	Adjusted Strength
1	Split Tensile	4.00 MPa (580 psi)	-----
2	Split Tensile	5.01 MPa (725 psi)	-----
3	Compression	44.7 MPa (6490 psi)	44.0 MPa (6380 psi)
4	Compression	49.3 MPa (7150 psi)	48.5 MPa (7040 psi)

Table 2.5. Chloride Content Test Results on Concrete Slab.

Sample No.	Depth	% Cl by Weight of Concrete	kg/m^3 (lbs/yd^3)
1A	25 mm (1")	0.027	0.63 (1.06)
1B	50 mm (2")	0.018	0.41 (0.71)
1C	75 mm (3")	Trace	< 0.36 (0.60)
1D	100 mm (4")	Trace	< 0.36 (0.60))
1E	126 mm (5")	Trace	< 0.36 (0.60)
2A	25 mm (1")	0.020	0.46 (0.78)
2B	50 mm (2")	Trace	< 0.36 (0.60)
2C	75 mm (3")	0.046	1.12 (1.88)
2D	100 mm (4")	0.025	0.58 (0.98)
2E	126 mm (5")	0.025	0.58 (0.98)
2F	152 mm (6")	0.025	0.58 (0.98)

Table 2.6. Properties of Recycled Coarse Aggregate

Property	**Recycled CA**	**ASTM Minimum**	**ASTM Maximum**
Sp. Gr. (SSD)	2.61	-	-
% Absorption	6.1	-	-
DRUW, kg/m^3 (pcf)	1480 (92.2)	-	-
Sieve Size	**% Passing**		
38.1 mm (1 1/2")	100.0	100	100
25.4 mm (1")	100.0	95	100
12.7 mm (1/2")	42.3	25	60
4.75 mm (#4)	3.1	0	10
2.36 mm (#8)	0	0	5

Table 2.7. Properties of Recycled Fine Aggregate

Property	Recycled FA	ASTM Minimum	ASTM Maximum
Sp. Gr. (SSD)	2.39	-	-
% Absorption	10.5	-	-
Fineness Modulus	2.58	-	-
Sieve Size	**% Passing**		
4.75 mm (#4)	100.0	95	100
2.36 mm (#8)	90.5	80	100
1.18 mm (#16)	68.4	50	85
600 μm (#30)	46.1	25	60
300 μm (#50)	26.0	10	30
150 μm (#100)	11.2	2	10

Table 2.8. Mixture Proportions for NCDOT Mix and NCSU Mix 1

	Quantity	
Materials	**NCDOT Mix**	**NCSU Mix 1**
Cement, kg/m^3 (pcy)	312 (526)	368 (620)
Coarse Aggregate, kg/m^3 (pcy)	1112 (1875)	1079 (1819)
Fine Aggregate, kg/m^3 (pcy)	722 (1217)	647 (1091)
Water, kg/m^3 (pcy)	148 (250)	173 (291)
AEA, L/m^3 (oz/yd^3)	0.17 (4.5)	0.17 (4.5)
AEA, L/m^3 (oz/cwt)	0.17 (0.86)	0.17 (0.73)
HRWR, L/m^3 (oz/yd^3)	1.20 (31)	3.48 (90)
HRWR, L/m^3 (oz/cwt)	1.20 (5.9)	3.48 (14.5)

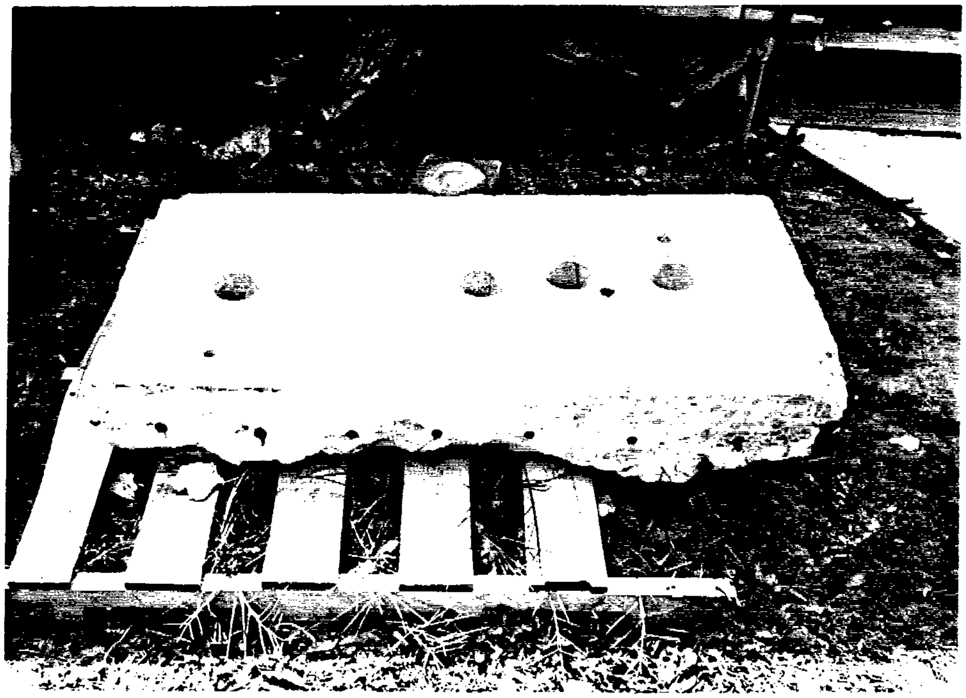


Figure 2.1 Test Section of Concrete Slab at NCDOT Materials & Tests Unit

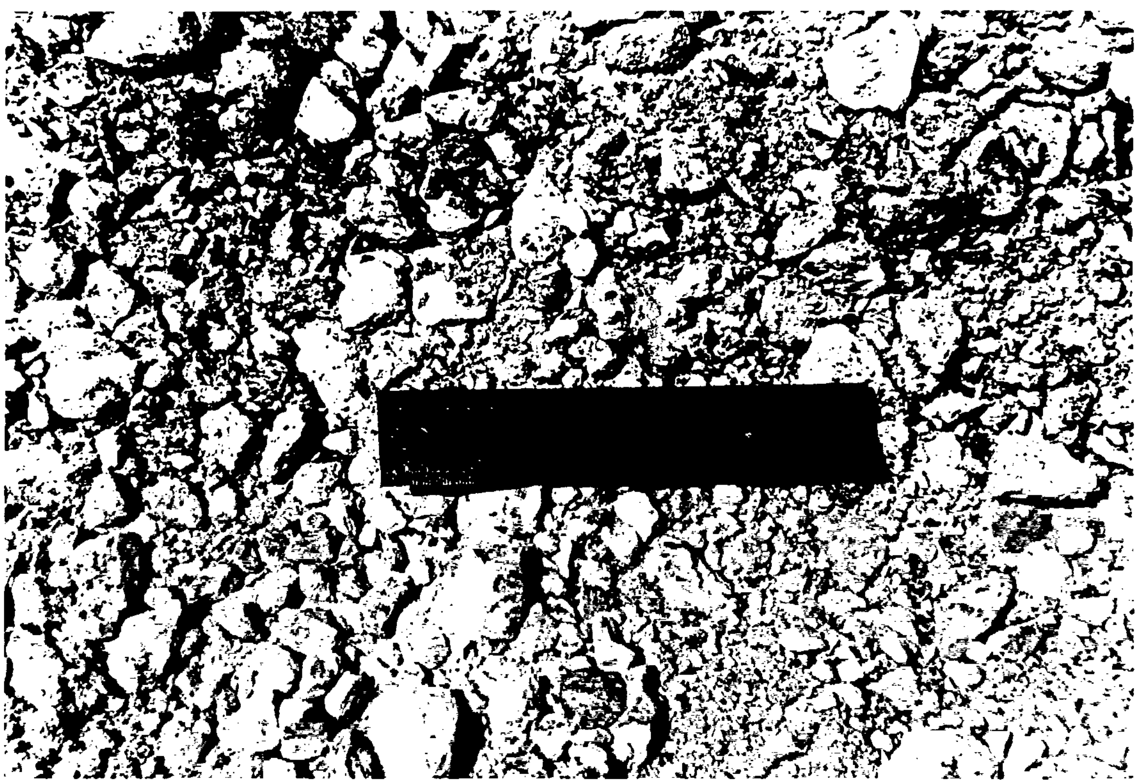


Figure 2.2 Photograph of Unsieved Recycled Aggregates

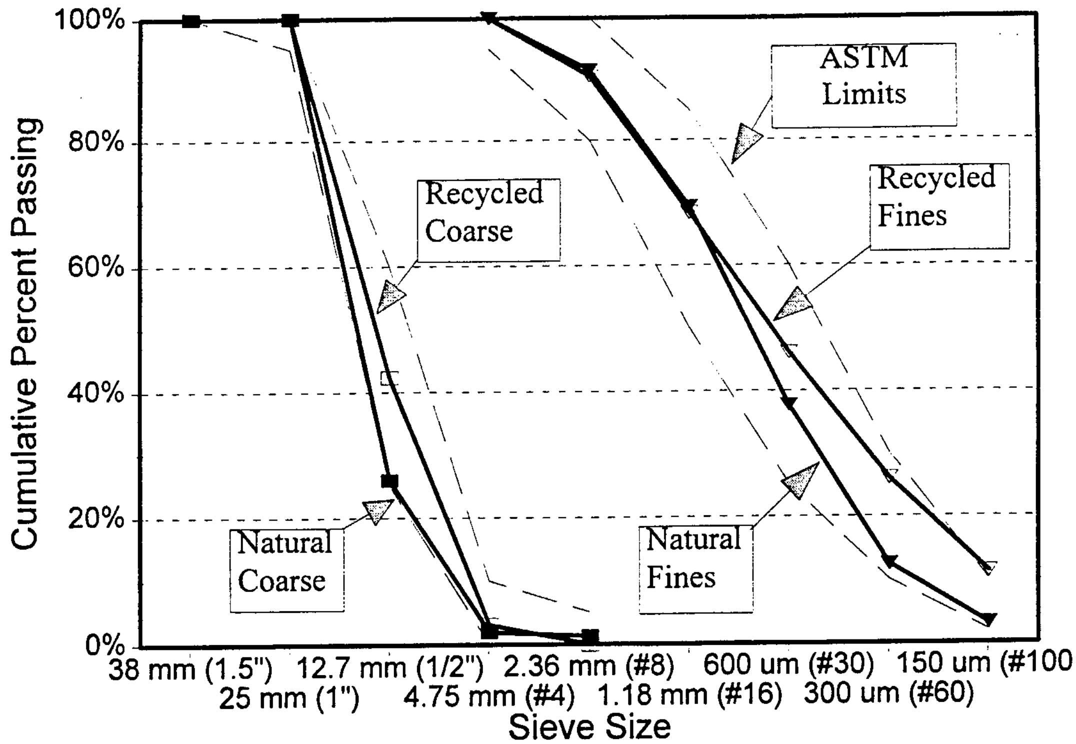


Figure 2.3 Grain Size Distribution of Natural and Recycled Aggregates

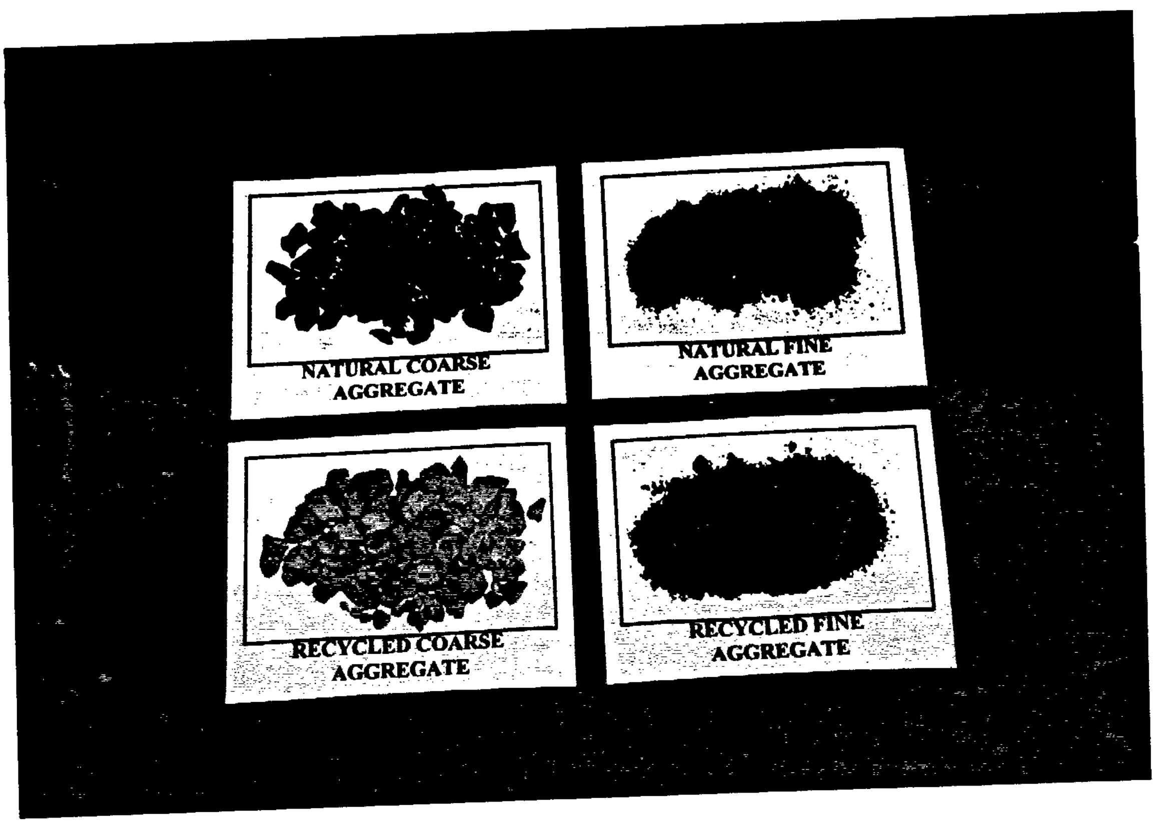


Figure 2.4 Photograph of Natural and Recycled Aggregates.

Figure 2.5 Laboratory Setup for Fabrication of Specimens

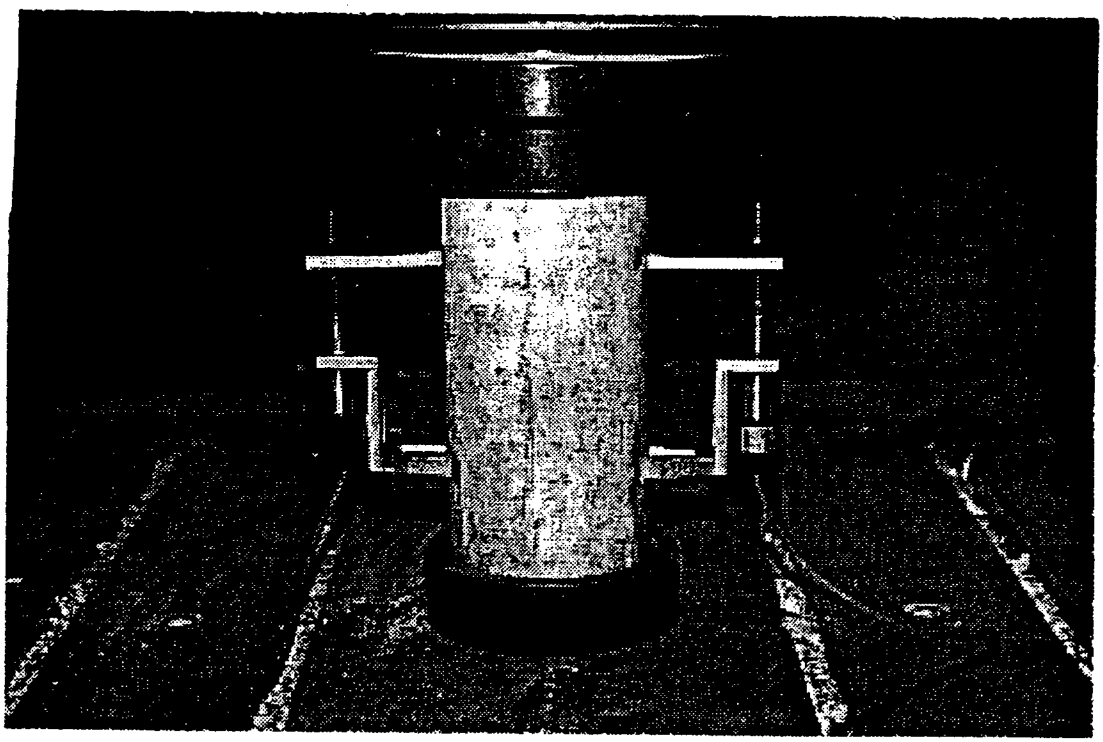


Figure 2.6 Compression and Elastic Modulus Test Setup

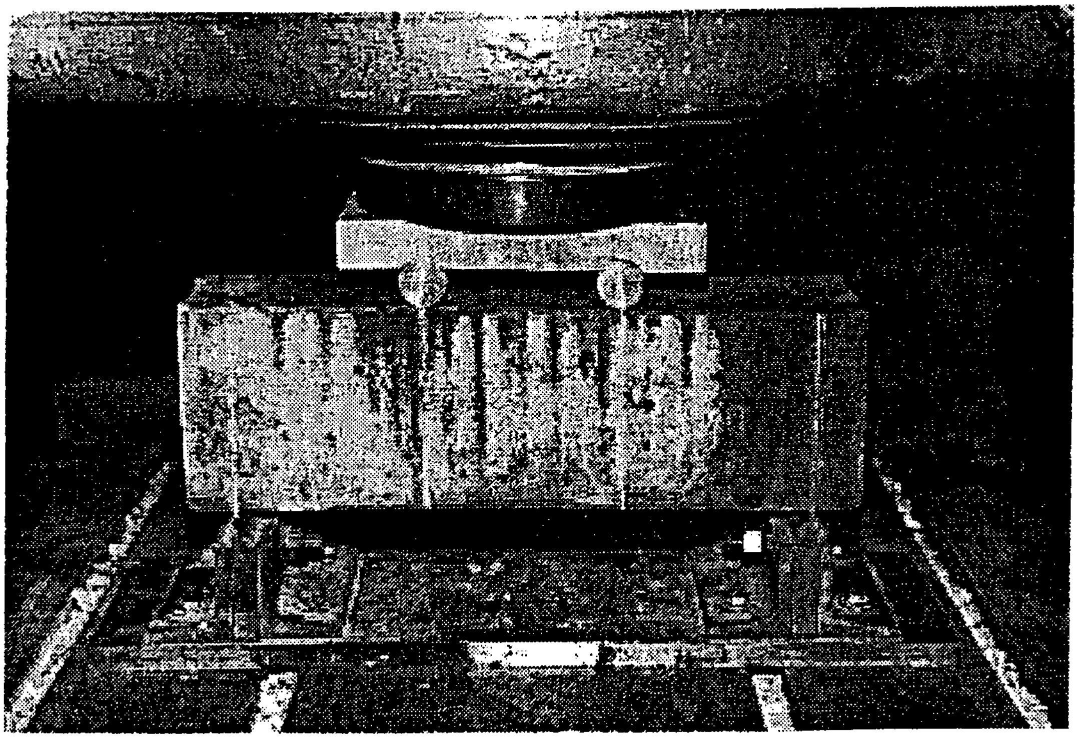


Figure 2.7 Flexural Beam Test Setup

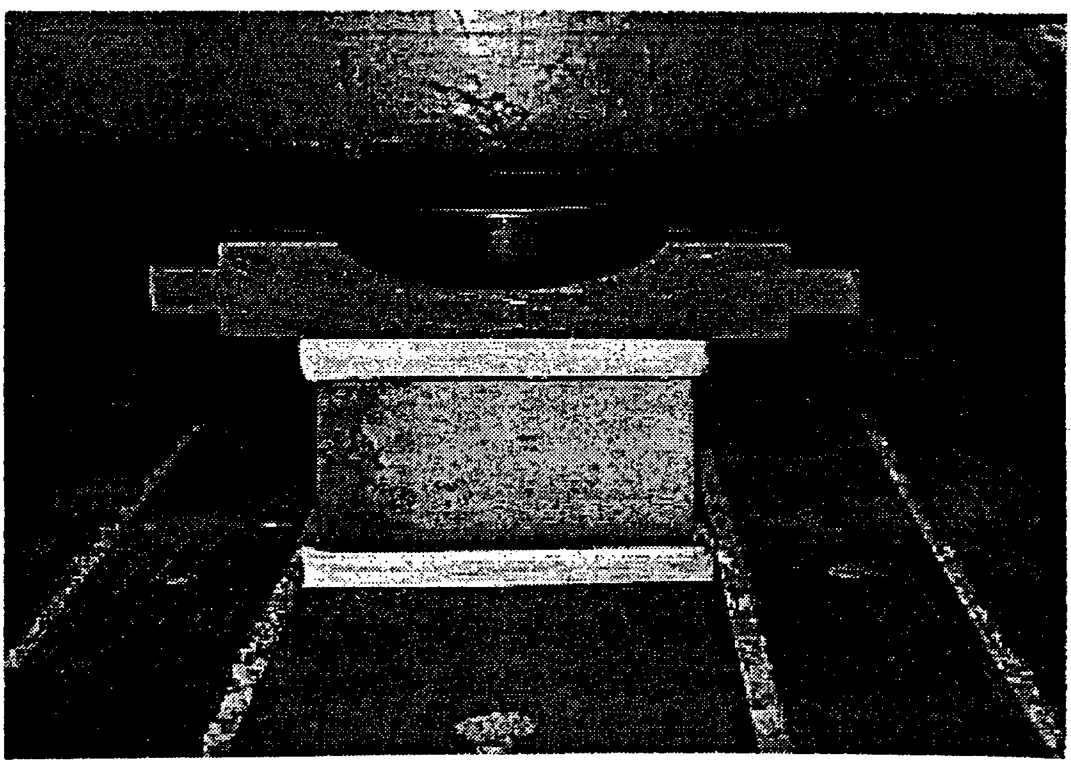


Figure 2.8 Splitting Tensile Test Setup

3. ANALYSIS OF TEST RESULTS

This chapter presents the results of the tests described in Chapter 2. The test results of fresh, or plastic, concrete were used as a measure of quality control and are presented first. The test results of hardened concrete including compressive strength, modulus of elasticity, flexural strength, and splitting tensile strength are presented next. Finally, the results of predictions using the empirical equations available in the AASHTO and ACI codes are compared with the experimental results.

3.1 Fresh Concrete Testing

3.1.1 Slump

Throughout the investigation, the w/c ratio of the mixture was kept at a constant value of 0.47. The slump of mixtures with recycled aggregates decreased as the percentage of recycled fine aggregate increased. To achieve the target slump, the amount of HRWR was increased as the percentage of recycled fine aggregate increased. The actual amounts HRWR used in each batch are shown in **Appendix B**. Without increasing the amount of HRWR, increasing the percentage of recycled fine aggregate would have resulted in a loss of workability.

3.1.2 Air Content

As with the slump, it was found that increasing the percentage of recycled fine aggregate resulted in a drop in the measured air content. To keep the air content within specified tolerances, an increase in the amount of AEA was required. The actual amounts of AEA used in each batch are presented in **Appendix B**. The concrete mixture with 100% recycled fine aggregate generally required more than twice the amount of AEA as the batch with 0% recycled fine aggregate. This problem was acute enough that in spite of extremely high dosages of AEA, the batch with 100% recycled coarse and fine aggregates did not meet the specified air content of 5.5% ± 1.5%.

3.1.3 Unit Weight

As the amount of recycled material, coarse or fine, increased, the unit weight of the concrete decreased. This decrease was expected since the specific gravity of the recycled aggregates was lower than that of the natural aggregates, and substitution was done on a volumetric basis. The unit weight of each batch tested is presented in **Appendix B**.

3.2 Hardened Concrete Testing

For the hardened concrete, the tests included compressive strength, modulus of elasticity, flexural strength, and splitting tensile strength.

3.2.1 Compressive Strength

The compressive strength tests were conducted at the ages of 14 days, 28 days, and 9 months. Three replicate cylinder specimens were tested for each test age. The results of the compression testing are summarized in Tables C.1, C.2, and C.3 of Appendix C shown in **Figure 3.1**. Each point on the graph is the average of three replicate cylinders. For concrete mixtures with 100% RCA, the average strength should be lower than for mixtures with 60% RCA, however, at 14 days and for concretes with high (more than 75% RFA), the results of 100% RCA concretes are higher than for 60% RCA concretes.

To determine the degree of strength reduction due to the recycled fine aggregate, the compressive strength of each mixture was normalized by dividing its strength by the strength of the companion mixture at the same age with all natural fine aggregate. **Figure 3.2** shows the normalized values for compressive strengths. From this figure, it can be seen that concrete strength decreases approximately in a linear fashion as the percentage replacement of natural fine aggregate with recycled fine aggregate increases. Furthermore, the overall strength reduction appears to be relatively insensitive to both age of the concrete and the amount of recycled fine aggregate used in the mixture. For concretes with 60% RCA and older than 14 days, the decrease in compressive strength for the mixtures with 100% recycled fines was approximately 25% to 30% of the mixtures with all natural fine aggregate.

3.2.2 Modulus of Elasticity

The modulus of elasticity tests were conducted at the ages of 14 days, 28 days, and 9 months. Three replicate cylinder specimens were tested for each test age. The results of the modulus of elasticity testing are summarized in Table C.4, C.5 and C.6 of Appendix C and shown in **Figure 3.3**. Each point on the graph is the average of three replicate cylinders.

As with compressive strength results, the elastic modulus values were normalized as a percentage of the appropriate mixture with no recycled fine aggregate. The normalized values are shown in **Figure 3.4**. All the mixtures made with 60% RCA show an overall decrease in elastic modulus with increasing percentage of recycled fine aggregate. As with the compressive strength, the decrease is sensitive to age of the concrete. For concrete with 60% RCA, the decrease in modulus of elasticity for the mixtures with 100% recycled fines was approximately 28% to 40% of the mixtures with all natural fine aggregate. It should be pointed out that the test data for concrete mixtures with 100% RCA and 50% RCA and 75% is apparently inconsistent with the general trend and could be disregarded.

3.2.3 Flexural Strength

The flexural strength tests were conducted at the ages of 14 days, 28 days, and 9 months. Three replicate beam specimens were tested for each test age. The results of the flexural strength testing are summarized in Tables C.7, C.8 and C.9 of Appendix C and shown in **Figure 3.5**.

The normalized values for the flexural strengths are shown in **Figure 3.6**. From this, the flexural strength can be seen to decrease with increasing percentage of recycled fine aggregate. For concrete with 60% RCA, the decrease in flexural strength for the mixtures with 100% recycled fines was approximately 15% to 20% of the mixtures with all natural fine aggregate.

3.2.4 Splitting Tensile Strength

The splitting tensile strength tests were conducted at the ages of 14 days, 28 days, and 9 months. Three replicate cylinder specimens were tested for each test age. The results of the splitting tensile strength testing are summarized in Tables C.10, C.11, and C.12 of Appendix C and shown in **Figure 3.7**. The normalized results are shown in **Figure 3.8**.

The splitting tensile strength results show a linear decrease in strength with increasing percentage of recycled fine aggregate. For concrete with 60% RCA, the overall decrease in splitting tensile strength is 18% to 27% of the mixtures with all natural fine aggregate.

3.3 Empirical Relationships

In order to facilitate the use of recycled aggregate concrete, the relationships between the various mechanical properties of hardened concrete must be understood. Of particular interest is whether the provisions of the current standard specifications which were developed for concretes with natural aggregates can also be used for concretes with recycled aggregates. This section compares the predicted results as per the equations available in AASHTO and ACI specifications to the experimental data.

3.3.1 Relationship for Modulus of Elasticity and Compressive Strength

For estimating the modulus of elasticity, the *AASHTO Guide for Design of Pavement Structures* [8] and ACI 318 *Building Code Requirements for Reinforced Concrete* [9] both suggest the following equation for normal weight concretes.

$$E_c = 4700\sqrt{f_c'} \qquad \textit{(SI Units)} \qquad \textit{(Eq. 3.3a)}$$

$$E_c = 57000\sqrt{f_c'} \qquad \textit{(US Customary Units)} \qquad \textit{(Eq. 3.3b)}$$

These equations are valid for concretes with unit weights between 1500 and 2500 kg/m^3 (90 to 155 pcf). The unit weight of the recycled aggregates concretes varied from 2200 to 2270 kg/m^3 (137 to 142 pcf).

To determine the validity of Equation 3.3 for recycled aggregate concrete, this equation was used to calculate the modulus of elasticity of each specimen at 28 days using the compressive strength values obtained from testing. The calculated values are shown in **Table 3.1**. Due to the variability in the data, it is difficult to determine whether Equation 3.3 is a reasonable predictor of the elastic modulus. A linear regression was performed on the data using a relationship of the form $E_c = C\sqrt{f_c}$. The results from this linear regression gave a $C = 4690$ for SI unit and a $C = 56500$ for US customary units. These values are very close to AASHTO [8] values. A plot of compressive strength vs. modulus of elasticity for both experimental and calculated values is shown in **Figure 3.9**.

3.3.2 Relationship for Flexural Strength

For estimating the flexural strength from the compressive strength, ACI 325, *Recommendations for Designing Prestressed Concrete Pavements* [10] suggests the following equation.

$$f_r = 0.75\sqrt{f_c'} \qquad \text{(SI Units)} \qquad \text{(Eq. 3.4a)}$$

$$f_r = 9\sqrt{f_c'} \qquad \text{(US Customary Units)} \qquad \text{(Eq. 3.4b)}$$

Another equation is suggested in section 2.5 of ACI 330, *Guide for Design and Construction of Concrete Parking Lots* [11], for estimating the flexural strength. This relationship is given in Equation 3.5.

$$f_r = 0.44 f_c'^{\,2/3} \qquad \text{(SI Units)} \qquad \text{(Eq. 3.5a)}$$

$$f_r = 2.3 f_c'^{\,2/3} \qquad \text{(US Customary Units)} \qquad \text{(Eq. 3.5b)}$$

ACI 318 [9] gives a relationship between flexural strength and compressive strength in Section 9.5.2.3. The equation specified is given below as Equation 3.6.

$$f_r = 0.7\sqrt{f_c'} \qquad \text{(SI Units)} \qquad \text{(Eq. 3.6a)}$$

$$f_r = 7.5\sqrt{f_c'} \qquad \text{(US Customary Units)} \qquad \text{(Eq. 3.6b)}$$

A conversion from US customary units to SI units gives a constant of 0.62 for equation 3.6a, which is rounded to 0.7 in the ACI 318 specifications [9].

A conversion from US customary units to SI units gives a constant of 0.62 for equation 3.6a, which is rounded to 0.7 in the ACI 318 specifications [9].

Values of flexural strengths were computed from the compressive strength values obtained during testing using Equations 3.4, 3.5, and 3.6. These values are given in **Table 3.2**. From the table, it can be seen that the ACI 330 and 325 equations give a higher estimated flexural strength, while the ACI 318 equation gives a lower estimate of the flexural strength. When using this type of relationship for design purposes, a conservative estimate of the flexural strength is desired. To accomplish this, a linear regression of the data was performed, using a relationship of the form $f_r = C\sqrt{f_c}$. For SI units, this gave a constant $C = 0.73$ and for US customary units $C = 8.8$. The results of this equation are given in **Table 3.2** and plotted in **Figure 3.10**. For the experimental data, a lower bound was determined with a coefficient $C = 0.65$ for SI units, and $C = 8$ for US customary units. For most design purposes, the lower bound estimate could be used as the maximum allowable design requirement.

3.3.3 Relationship for Splitting Tensile Strength

For estimating the splitting tensile strength, ACI 318 Section R11.2.1.1 [9] suggests the following equation.

$$f_{ct} = \sqrt{f_c'}/1.8 \qquad \text{(SI Units)} \qquad \text{(Eq. 3.7a)}$$

$$f_{ct} = 6.7\sqrt{f_c'} \qquad \text{(US Customary Units)} \qquad \text{(Eq. 3.7b)}$$

Using Equation 4.7, the flexural strength splitting tensile strengths were calculated from the compressive strength values obtained from testing. The results are shown in **Table 3.3**. As before, a linear regression was also performed on the data, using a relationship of the form $f_{ct} = C\sqrt{f_c}$. The results from the regression gave a value of $C = 0.54$ for SI units and $C = 6.5$ for US customary units. Estimated splitting tensile strengths were then recalculated based on compressive strength values. The results of the linear regression analysis are shown in **Table 3.3**, and plotted in **Figure 3.11**. From this graph, it can be seen that the ACI 318 equation is slightly unconservative in estimating the splitting tensile strength of recycled aggregate concrete. A more conservative value would be appropriate for design purposes. A value of $C = 0.45$ for SI units and $C = 5.5$ for US customary units represents the lower bound for the data.

Of more interest in the design of pavements is the relationship between flexural strength and splitting tensile strength. The *AASHTO Guide for Design of Pavement Structures* [8] states that the splitting tensile strength may be taken as 86% of the flexural strength. To verify this for recycled aggregate concretes, splitting tensile strengths were calculated based on flexural strength data. The results are shown in **Table 3.3**. From **Figure 3.12**, it can be seen that AASHTO's recommendation overestimates the values of splitting tensile strength. Therefore a linear regression was performed based on a relationship $f_{ct} = C\, f_r$. The results of the regression gave a $C = 0.78$ for both SI and US customary units as shown in **Figure 3.12**.

Table 3.1A Comparison Between Experimental and Calculated Elastic Moduli at 28 days, SI Units

Mix ID	Comp. Strength (MPa)	Exp. Modulus (10^3 MPa)	AASHTO & ACI Calc. Modulus (10^3 MPa)	Calc. Modulus using $E_c = C\ f_c'$* (10^3 MPa)
B-60-0	37.4	31.9	28.7	28.7
	36.3	31.6	28.3	28.2
	33.9	28.5	27.4	27.3
B-60-25	31.9	29.9	26.5	26.5
	31.6	31.4	26.4	26.4
	31.2	28.4	26.2	26.2
B-60-50	30.2	26.2	25.8	25.8
	30.1	27.6	25.8	25.7
	29.9	27.0	25.7	25.6
B-60-75	28.2	21.4	25.0	24.9
	28.0	23.3	24.9	24.8
	27.6	24.0	24.7	24.7
B-60-100	26.8	22.1	24.3	24.3
	24.6	21.7	23.3	23.3
B-100-0	35.8	21.2	28.1	28.1
	37.6	23.2	28.8	28.8

* C = 4690 computed from regression analysis of the experimental data.

Table 3.1B Comparison Between Experimental and Calculated Elastic Moduli at 28 days, US Customary Units

Mix ID	Comp. Strength (psi)	Exp. Modulus (10^6 psi)	AASHTO & ACI Calc. Modulus (10^6 psi)	Calc. Modulus using $E_c = C\ f_c'$ * (10^6 psi)
B-60-0	5420	4.62	4.20	4.16
	5260	4.58	4.13	4.10
	4920	4.14	4.00	3.96
B-60-25	4620	4.34	3.87	3.84
	4590	4.56	3.86	3.83
	4520	4.12	3.83	3.80
B-60-50	4380	3.80	3.77	3.74
	4370	4.00	3.77	3.73
	4330	3.91	3.75	3.72
B-60-75	4090	3.11	3.65	3.61
	4060	3.38	3.63	3.60
	4010	3.48	3.61	3.58
B-60-100	3890	3.21	3.56	3.52
	3570	3.15	3.41	3.38
B-100-0	5200	3.07	4.11	4.07
	5460	3.36	4.21	4.17

* C = 56500 computed from regression analysis of the experimental data.

Table 3.2A Comparison Between Experimental and Calculated Flexural Strengths at 28 days, SI Units

Mix ID	Comp. Strength (MPa)	Exp. Flexural Strength (MPa)	ACI 330 Flexural Strength (MPa)	ACI 325 Flexural Strength (MPa)	ACI 318 Flexural Strength (MPa)	Calculated Flex. Str. using $f_r = C f_c'$ * (MPa)
B-60-0	37.4	4.16	4.28	4.58	4.92	4.28
	36.3	4.45	4.22	4.52	4.82	4.22
	33.9	3.86	4.08	4.37	4.61	4.08
B-60-25	31.9	3.83	3.95	4.23	4.42	3.95
	31.6	4.03	3.94	4.22	4.40	3.94
	31.2	3.96	3.91	4.19	4.36	3.91
B-60-50	30.2	3.72	3.85	4.12	4.27	3.85
	30.1	3.83	3.84	4.12	4.26	3.84
	29.9	4.14	3.82	4.10	4.23	3.82
B-60-75	28.2	3.83	3.72	3.98	4.08	3.72
	28.0	3.55	3.70	3.97	4.06	3.70
	27.6	3.69	3.68	3.94	4.02	3.68
B-60-100	27.2	3.58	3.65	3.91	3.98	3.65
	26.8	3.55	3.63	3.88	3.94	3.63
	24.6	3.48	3.47	3.72	3.72	3.47
B-100-0	37.6	4.17	4.29	4.60	4.94	4.29
	35.8	4.14	4.19	4.49	4.78	4.19
	33.1	4.14	4.03	4.31	5.54	4.03

- C = 0.73 computed from regression analysis of the experimental data.

Table 3.2B Comparison Between Experimental and Calculated Flexural Strength at 28 days, US Customary Units

Mix ID	Comp. Strength (psi)	Exp. Flexural Strength (psi)	ACI 330 Flexural Strength (psi)	ACI 325 Flexural Strength (psi)	ACI 318 Flexural Strength (psi)	Calculated Flex. Str. using $f_r = C\,f_c'$ * (psi)
B-60-0	5420	603	552	663	710	618
	5260	645	544	653	696	609
	4920	560	526	631	665	589
B-60-25	4620	555	510	612	638	571
	4590	585	508	610	635	569
	4520	575	504	605	629	565
B-60-50	4380	540	496	596	616	556
	4370	555	496	595	615	555
	4330	600	494	592	611	553
B-60-75	4090	555	480	576	588	537
	4060	515	478	573	585	535
	4010	535	475	570	581	532
B-60-100	3950	520	471	566	575	528
	3890	515	468	561	569	524
	3570	505	448	538	537	502
B-100-0	5460	605	554	665	713	621
	5200	600	541	649	690	606
	4800	600	520	624	654	589

* C = 8.8 computed from regression anaylsis of the experimental data.

Table 3.3A Comparison Between Experimental and Calculated Splitting Tensile Strengths at 28 days, SI Units

Mix ID	Comp. Str. (MPa)	Exp. Tensile Str. (MPa)	ACI 318 Tensile Str. (MPa)	Tensile Strength using $f_{ct} = C_1 f_c'$* (MPa)	Flexural Strength (MPa)	AASHTO Tensile Strength (MPa)	Tensile Strength using $f_{ct} = C_2 f_r$† (MPa)
B-60-0	37.4	3.58	3.58	3.24	4.16	2.75	3.30
	36.3	2.90	3.82	3.47	4.45	2.71	3.25
	33.9	3.65	3.32	3.01	3.86	2.62	3.14
B-60-25	31.9	3.24	3.29	2.98	3.83	2.54	3.05
	31.6	3.34	3.47	3.15	4.03	2.53	3.04
	31.2	3.24	3.41	3.09	3.96	2.51	3.01
B-60-50	30.2	2.55	3.20	2.90	3.72	2.47	2.97
	30.1	3.45	3.29	2.98	3.83	2.47	2.96
	29.9	2.90	3.56	3.23	4.14	2.46	2.95
B-60-75	28.2	2.86	3.29	2.98	3.83	2.39	2.87
	28.0	2.52	3.05	2.77	3.55	2.38	2.86
	27.6	2.86	3.17	2.88	3.69	2.37	2.84
B-60-100	27.2	3.03	3.08	2.80	3.58	2.35	2.82
	26.8	2.65	3.05	2.77	3.55	2.33	2.80
	24.6	2.52	2.99	2.72	3.48	2.23	2.68
B-100-0	37.6	2.86	3.59	3.25	4.17	2.76	3.31
	35.8	3.34	3.56	3.23	4.14	2.69	3.23
	33.1	3.10	3.56	3.23	4.14	2.59	3.11

* $C_1 = 0.54$ computed from regression anaylsis of the experimental data.
† $C_2 = 0.78$ computed from regression analysis of the experimental data.

Table 3.3B Comparison Between Experimental and Calculated Splitting Tensile Strengths at 28 days, US Customary Units

Mix ID	Comp. Str. (psi)	Exp. Tensile Str. (psi)	ACI 318 Tensile Str. (psi)	Tensile Strength using $f_{ct} = C_1 f_c'$* (psi)	Flexural Strength (psi)	AASHTO Tensile Strength (psi)	Tensile Strength using $f_{ct} = C_2 f_r$† (psi)
B-60-0	5420	520	493	405	603	519	470
	5260	420	486	399	645	555	503
	4920	530	470	386	560	482	437
B-60-25	4620	470	455	374	555	477	433
	4590	485	454	373	585	503	456
	4520	470	450	370	575	495	449
B-60-50	4380	370	443	364	540	464	421
	4370	500	443	364	555	477	433
	4330	420	441	362	600	516	468
B-60-75	4090	415	428	352	555	477	433
	4060	365	427	350	515	443	402
	4010	415	424	348	535	460	417
B-60-100	3950	440	421	346	520	447	406
	3890	385	418	343	515	443	402
	3570	365	400	329	505	434	394
B-100-0	5460	415	495	406	605	520	472
	5200	485	483	397	600	516	468
	4800	450	464	381	600	516	468

* C_1 = 6.5 computed from regression anaylsis of the experimental data.
† C_2 = 0.78 computed from regression analysis of the experimental data, SI Units

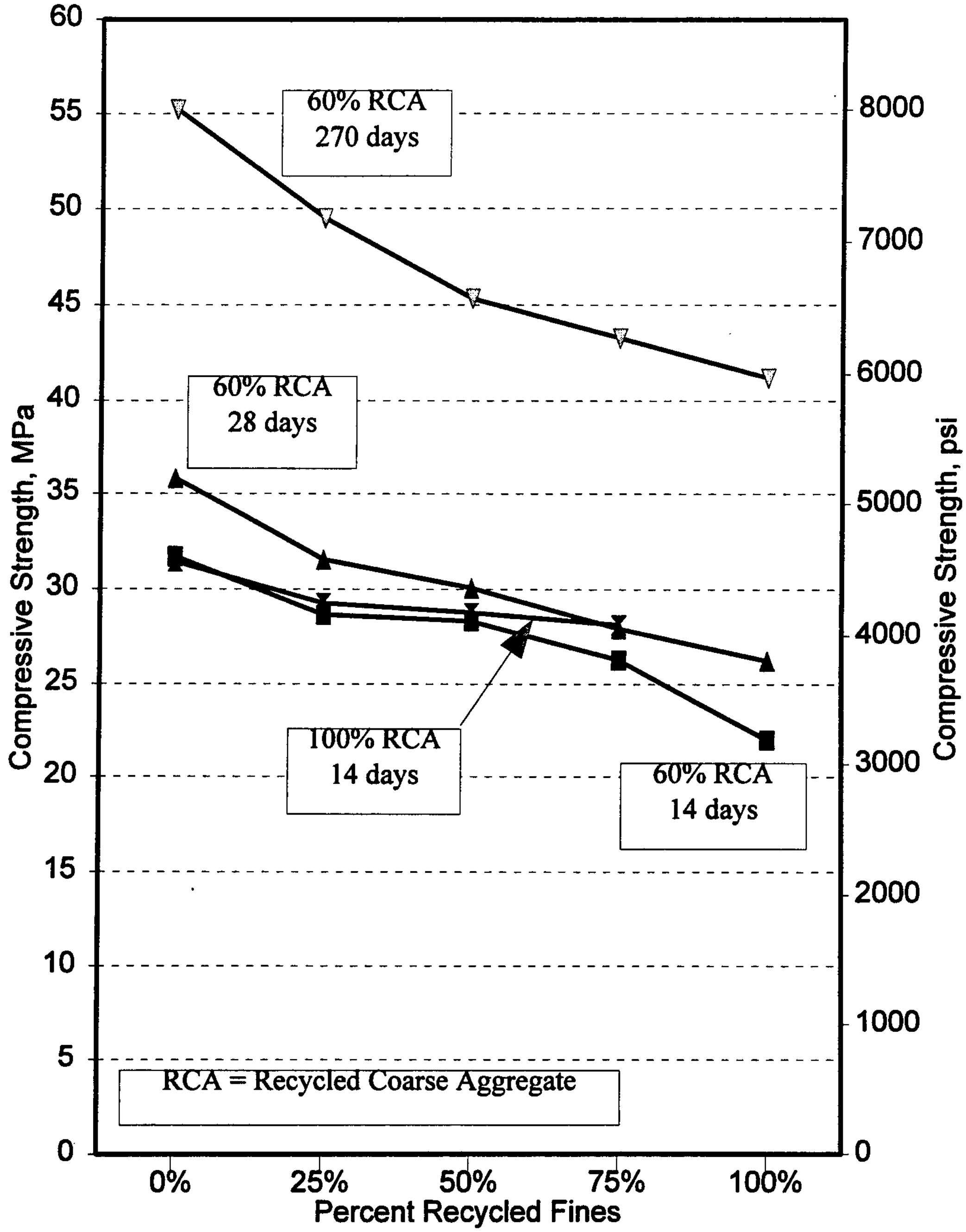


Figure 3.1. Average Compressive Strength vs. Percent Recycled Fine Aggregate

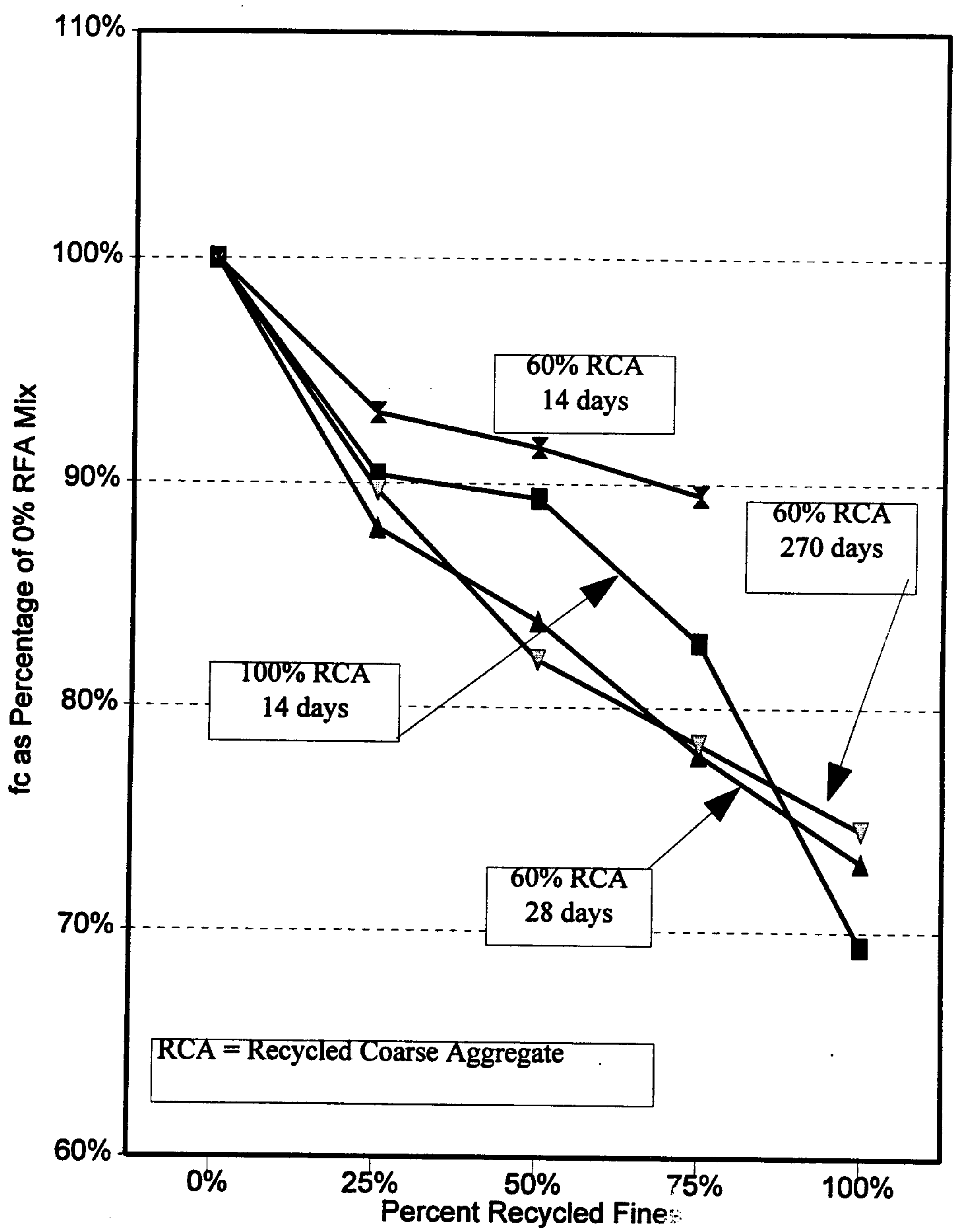


Figure 3.2. Normalized Compressive Strength vs. Percent Recycled Fine Aggregate

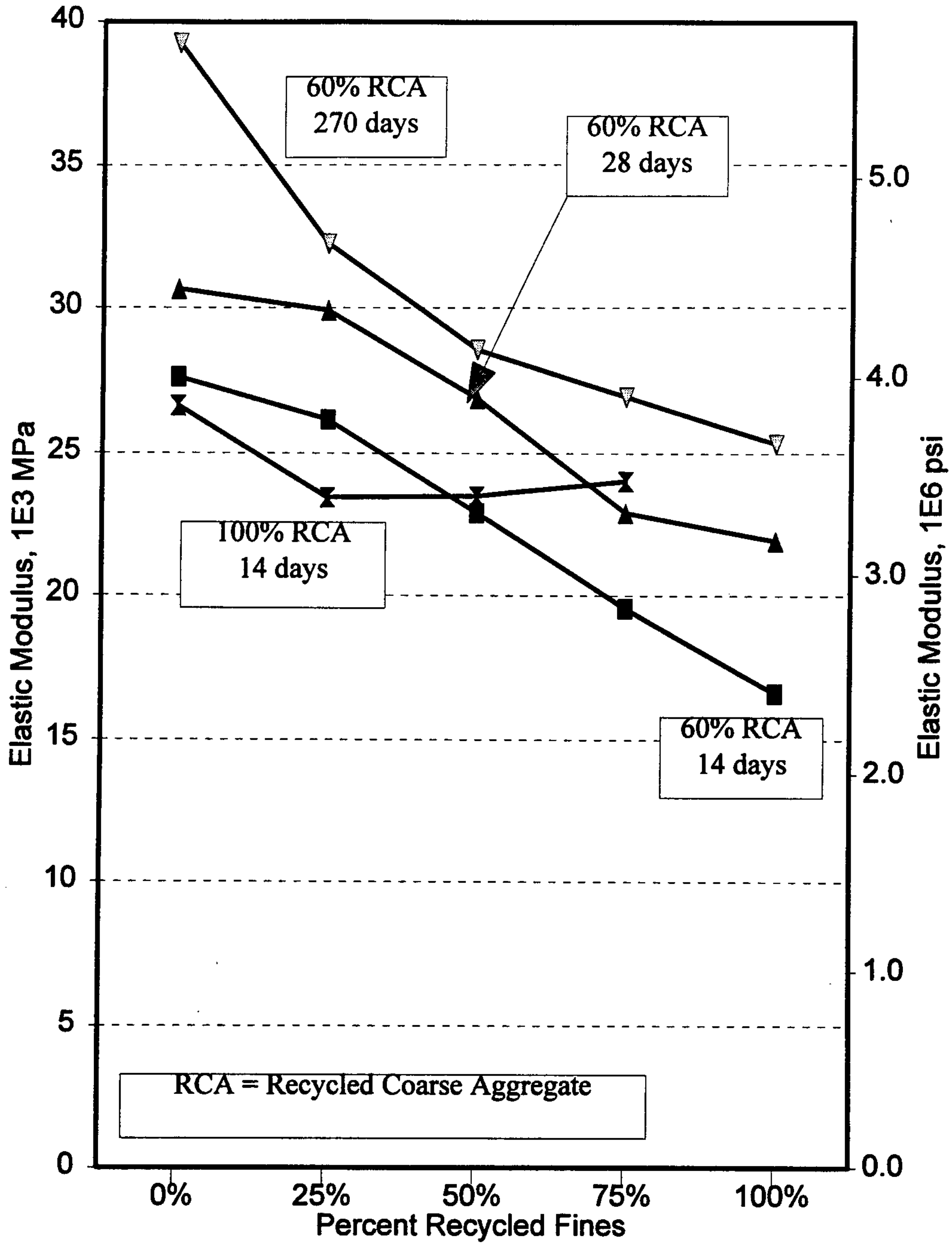


Figure 3.3. Average Elastic Modulus vs. Percent Recycled Fine Aggregate

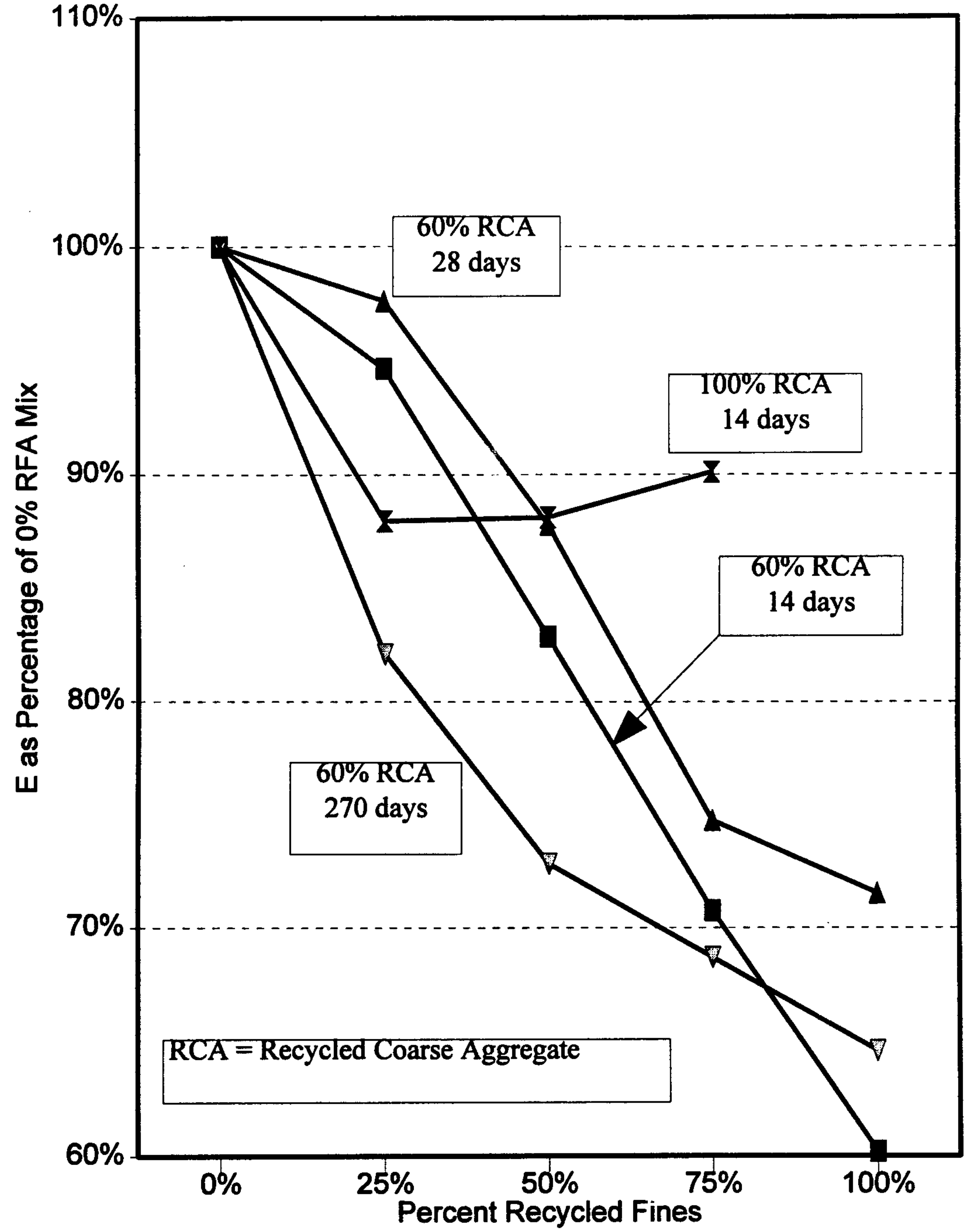


Figure 3.4. Normalized Elastic Modulus vs. Percent Recycled Fine Aggregate

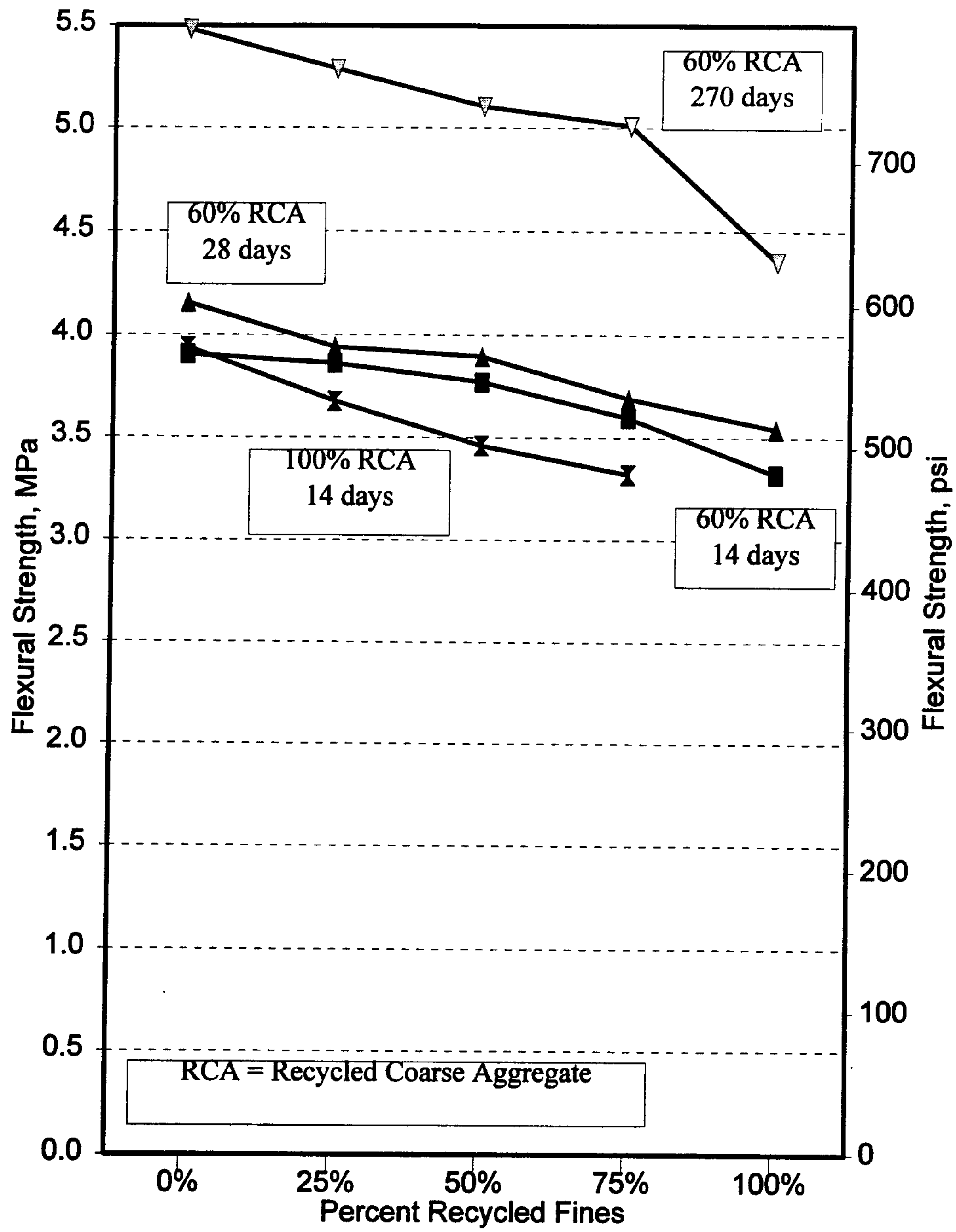


Figure 3.5. Average Flexural Strength vs. Percent Recycled Fine Aggregate

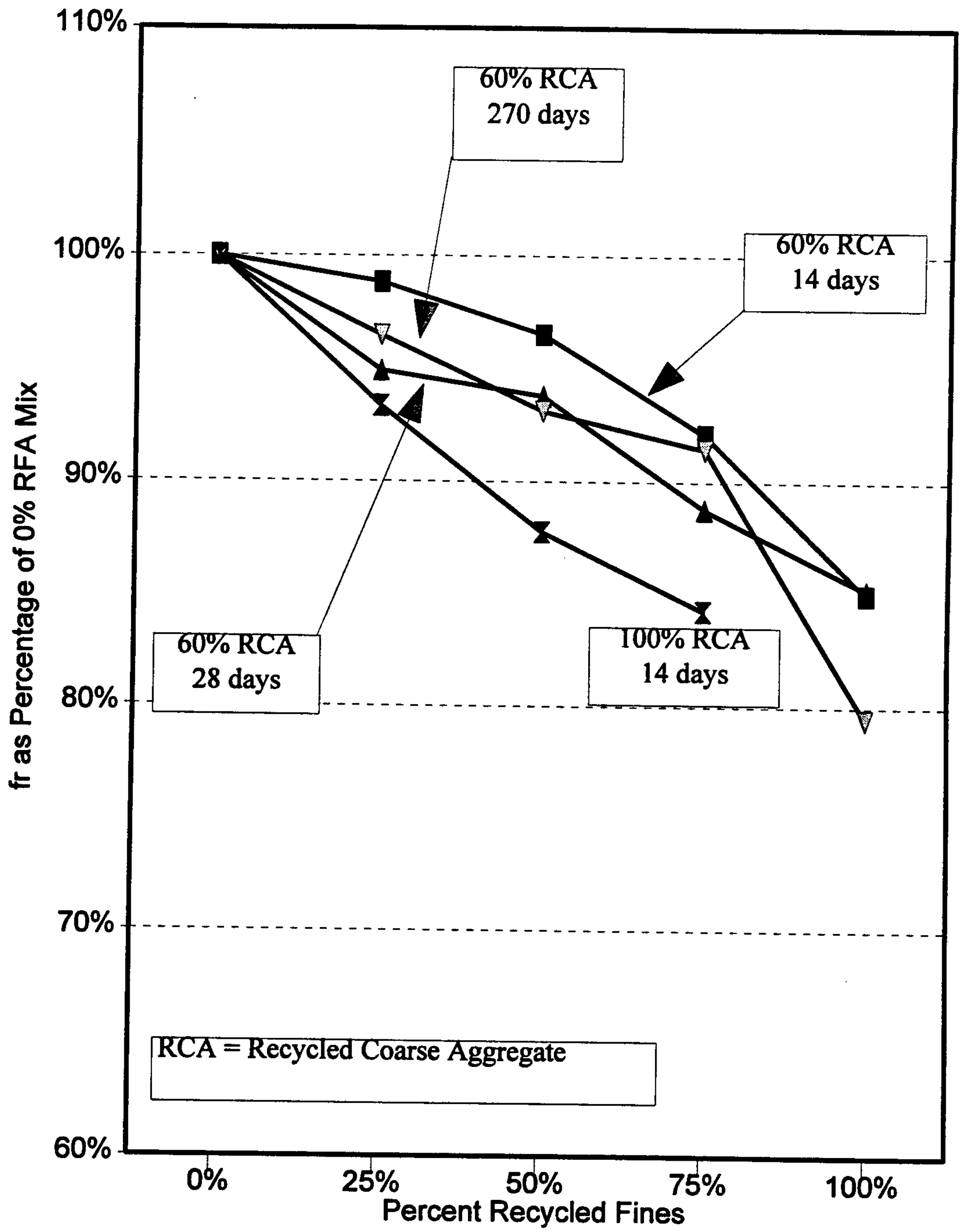


Figure 3.6. Normalized Flexural Strength vs. Percent Recycled Fine Aggregate

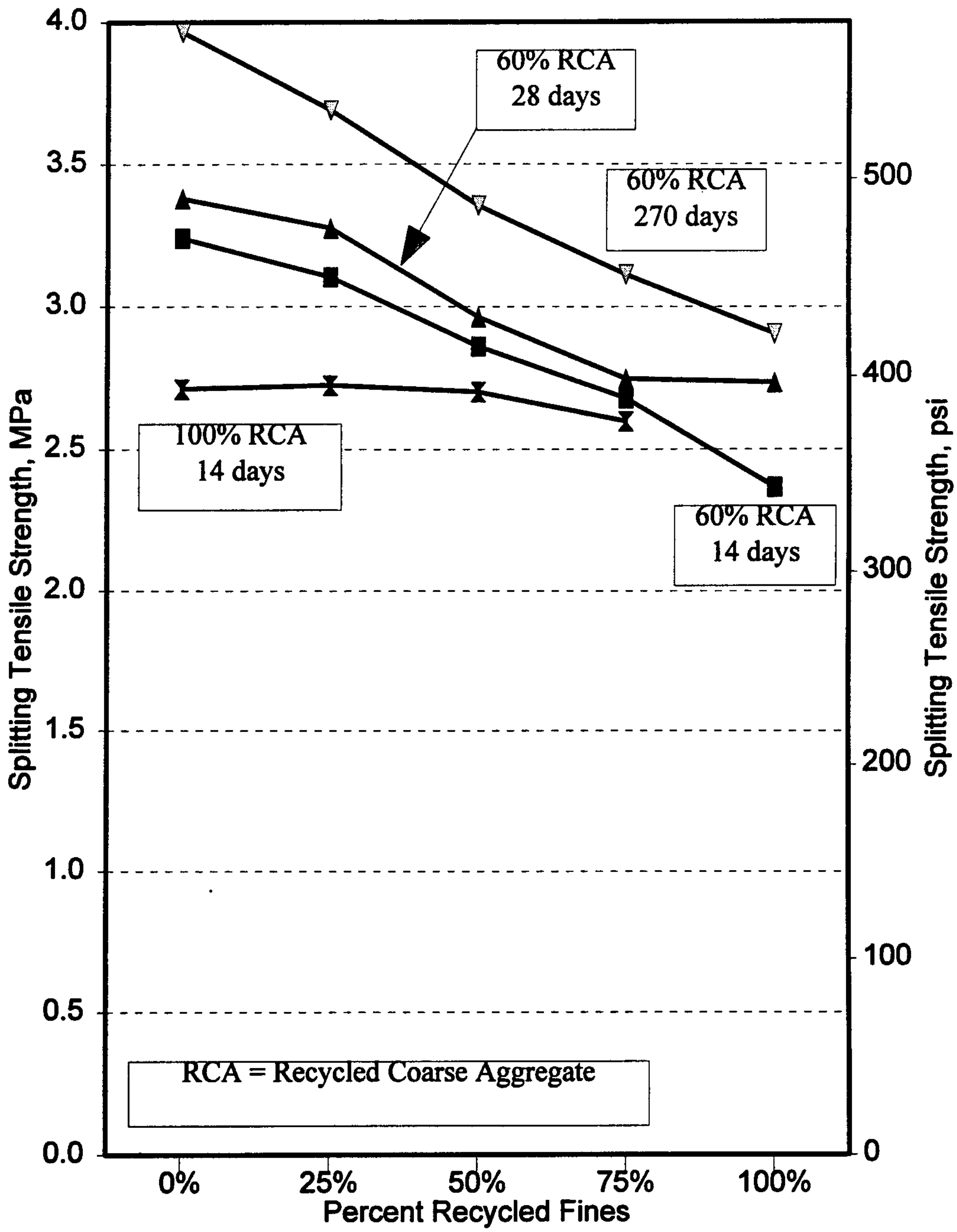


Figure 3.7. Average Splitting Tensile Strength vs. Percent Recycled Fine Aggregate

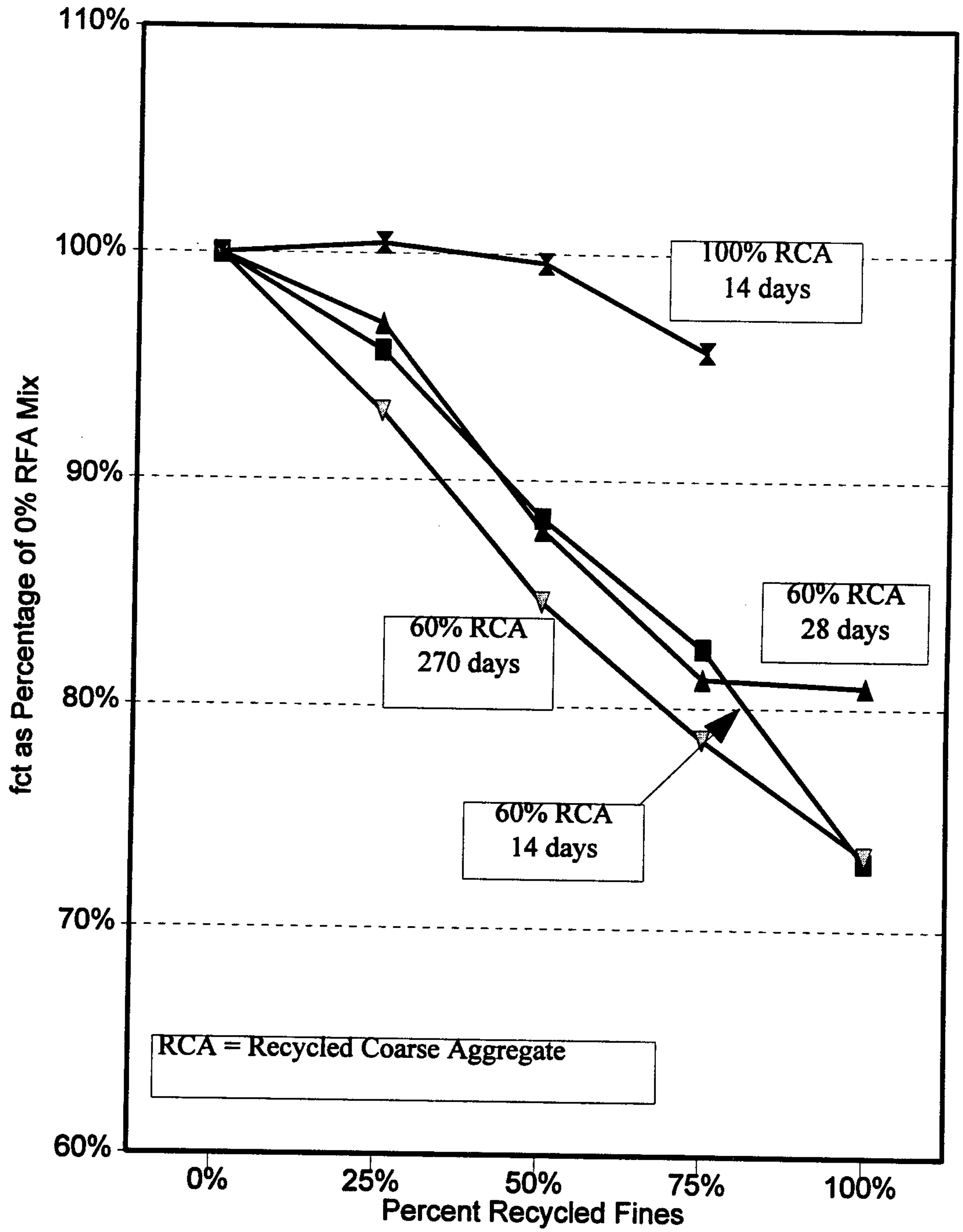


Figure 3.8. Normalized Splitting Tensile Strength vs. Percent Recycled Fine Aggregate

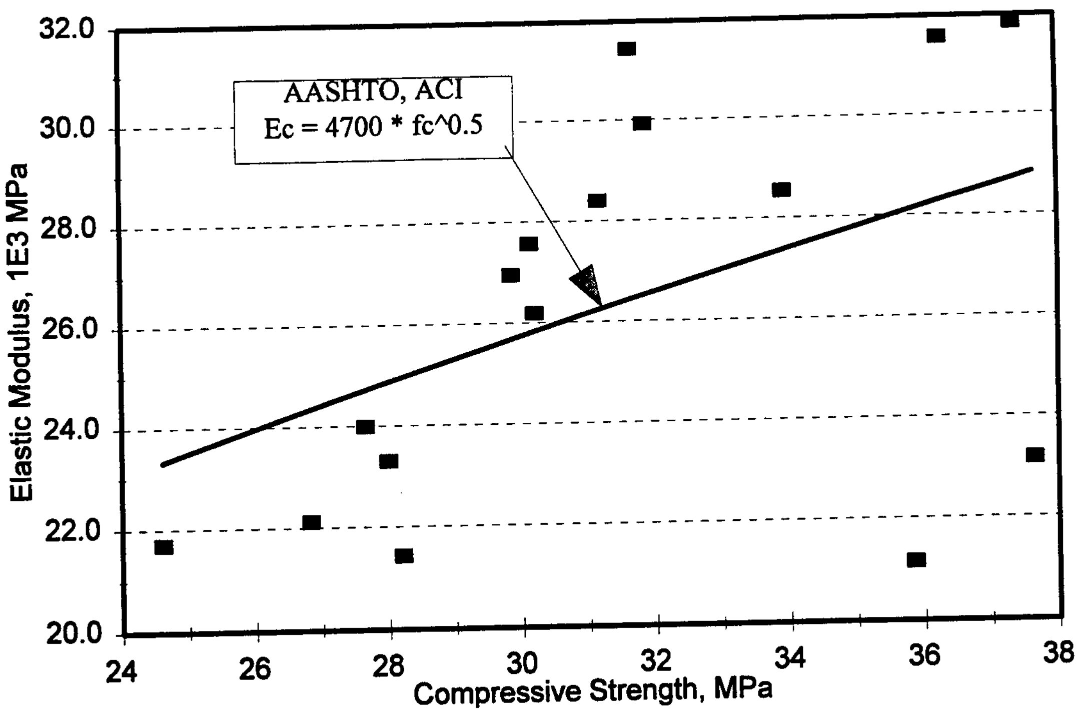


Figure 3.9A Plot of Compressive Strength vs. Experimental and Calculated Elastic Moduli, SI Units at 28 days

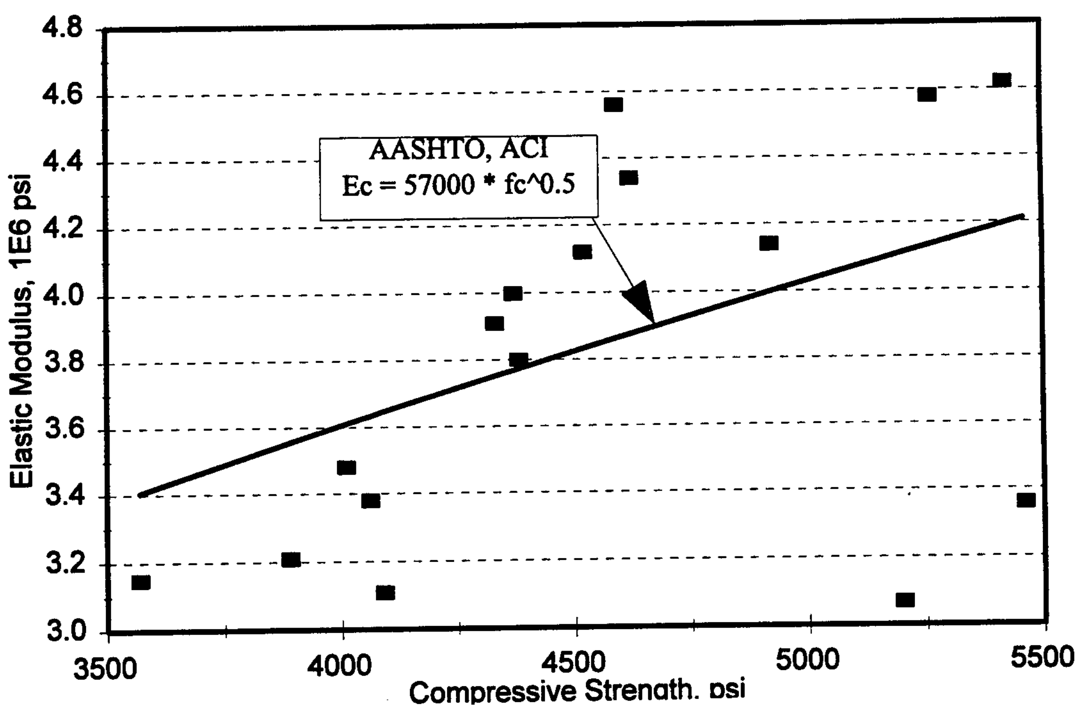


Figure 3.9B Plot of Compressive Strength vs. Experimental and Calculated Elastic Moduli, US Customary Units at 28 days

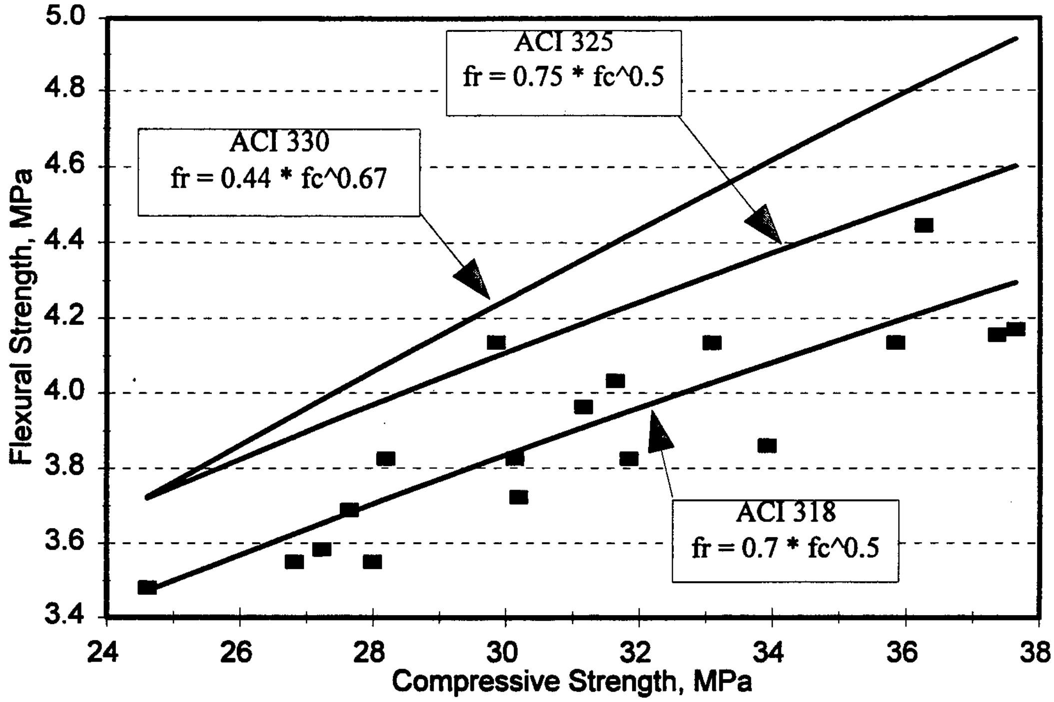


Figure 3.10A Plot of Compressive Strength vs. Experimental and Calculated Flexural Strengths, SI Units

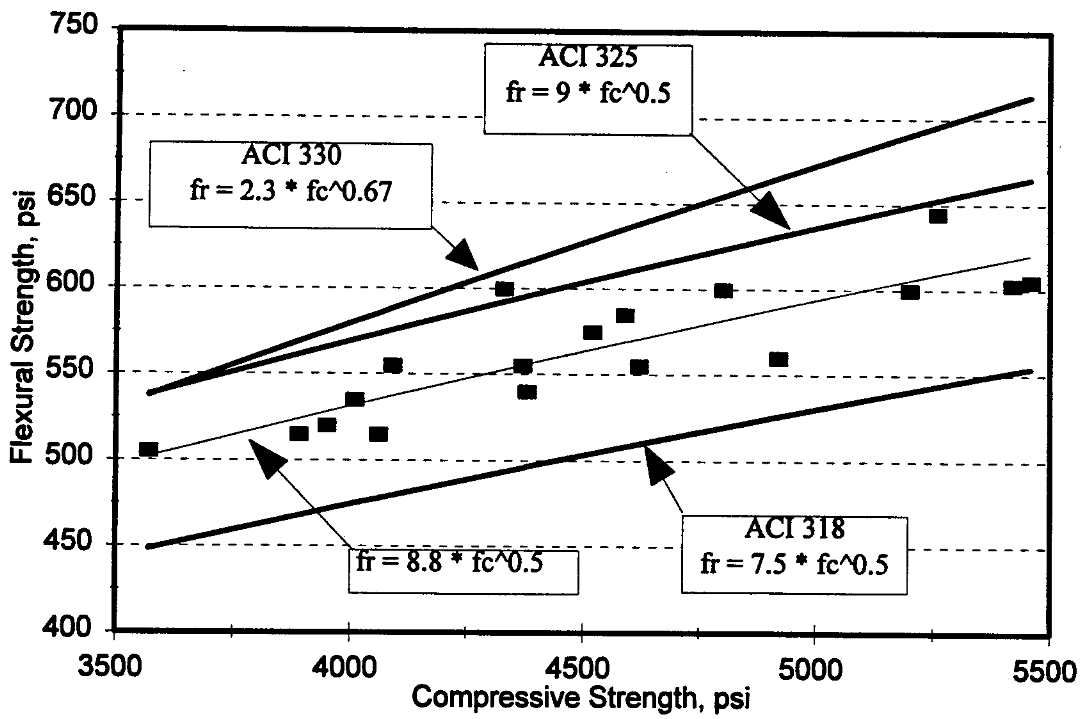


Figure 3.10B Plot of Compressive Strength vs. Experimental and Calculated Flexural Strengths, US Customary Units.

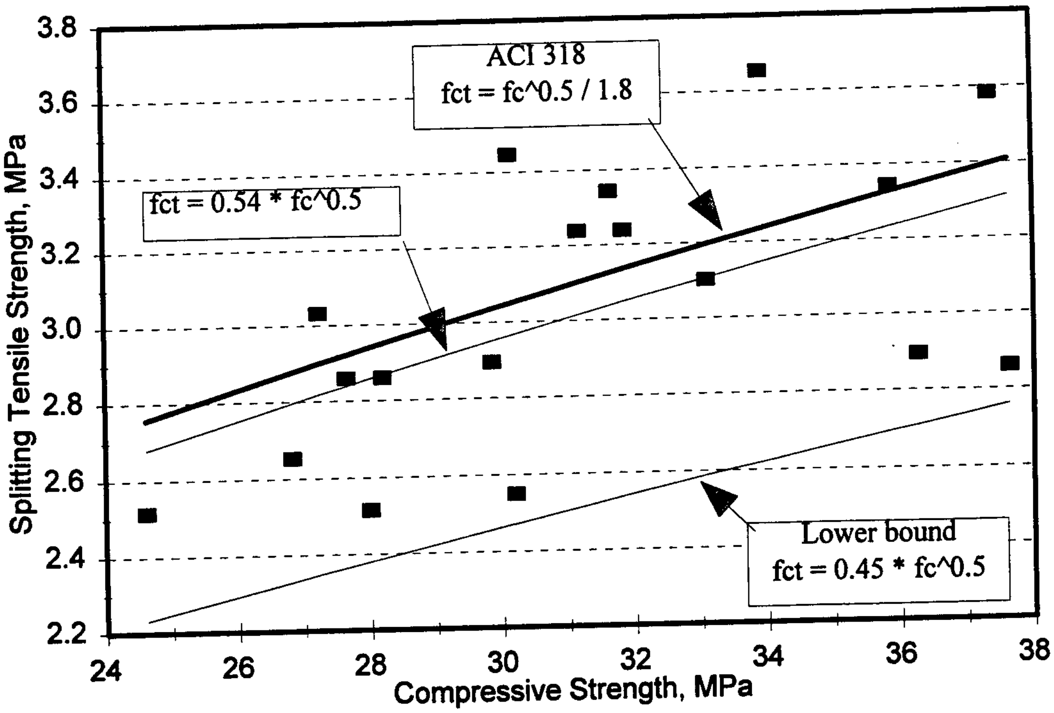


Figure 3.11A Plot of Compressive Strength vs. Experimental and Calculated Splitting Tensile Strengths, SI Units

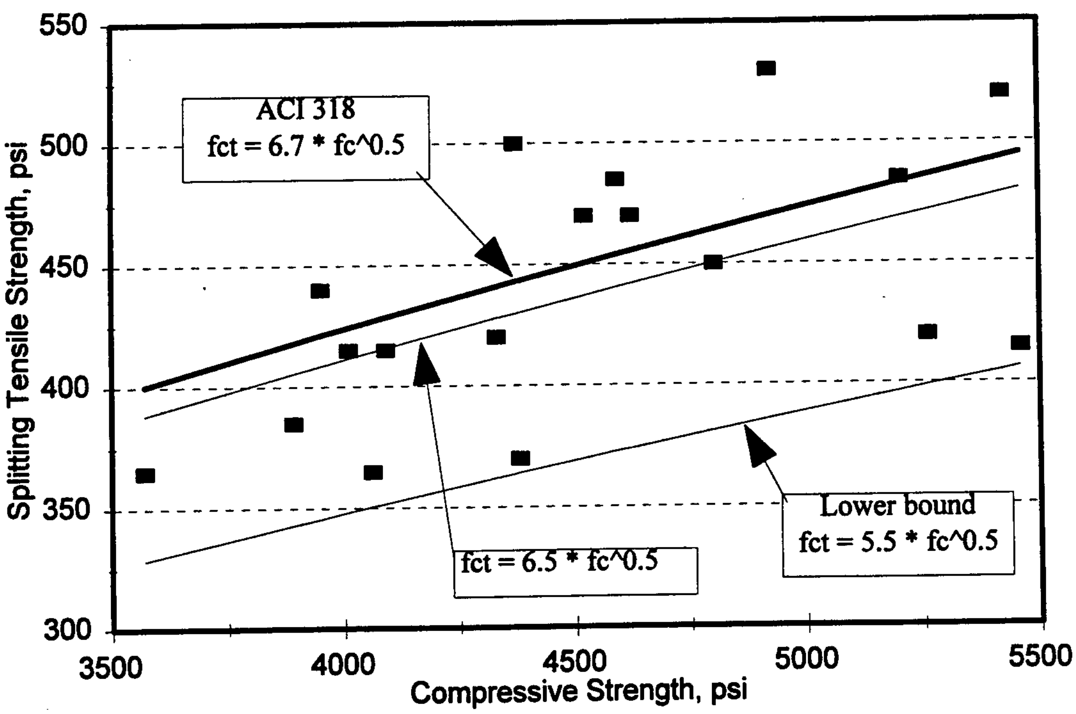


Figure 3.11B Plot of Compressive Strength vs. Experimental and Calculated Splitting Tensile Strengths, US Customary Units

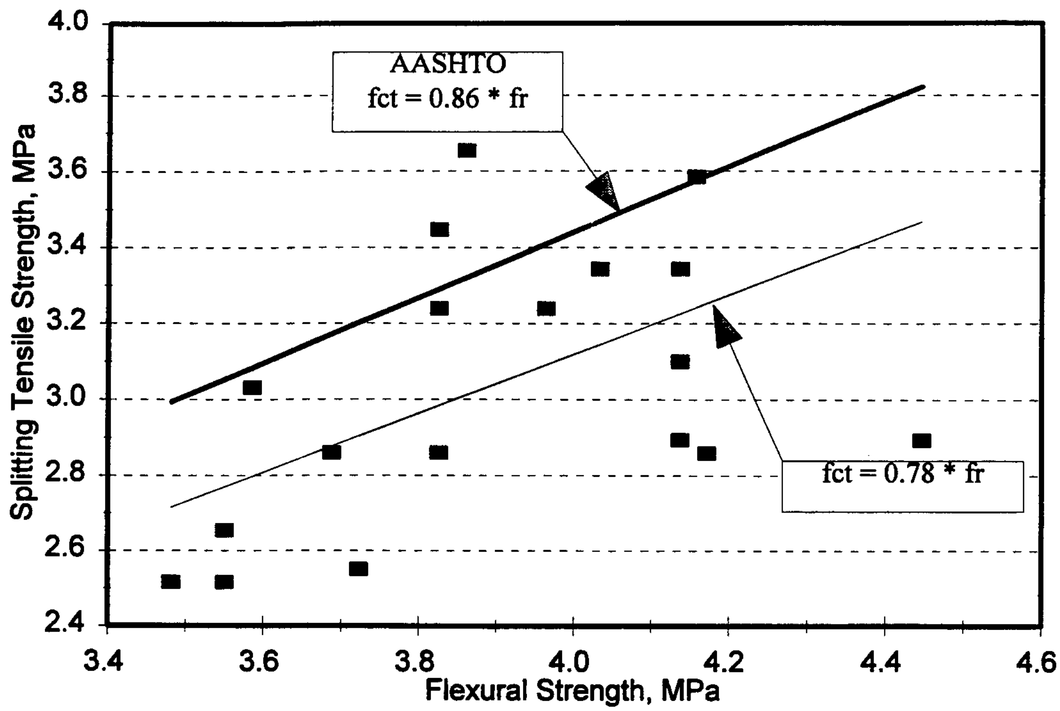


Figure 3.12A Plot of Flexural Strength vs. Experimental and Calculated Splitting Tensile Strengths, SI Units

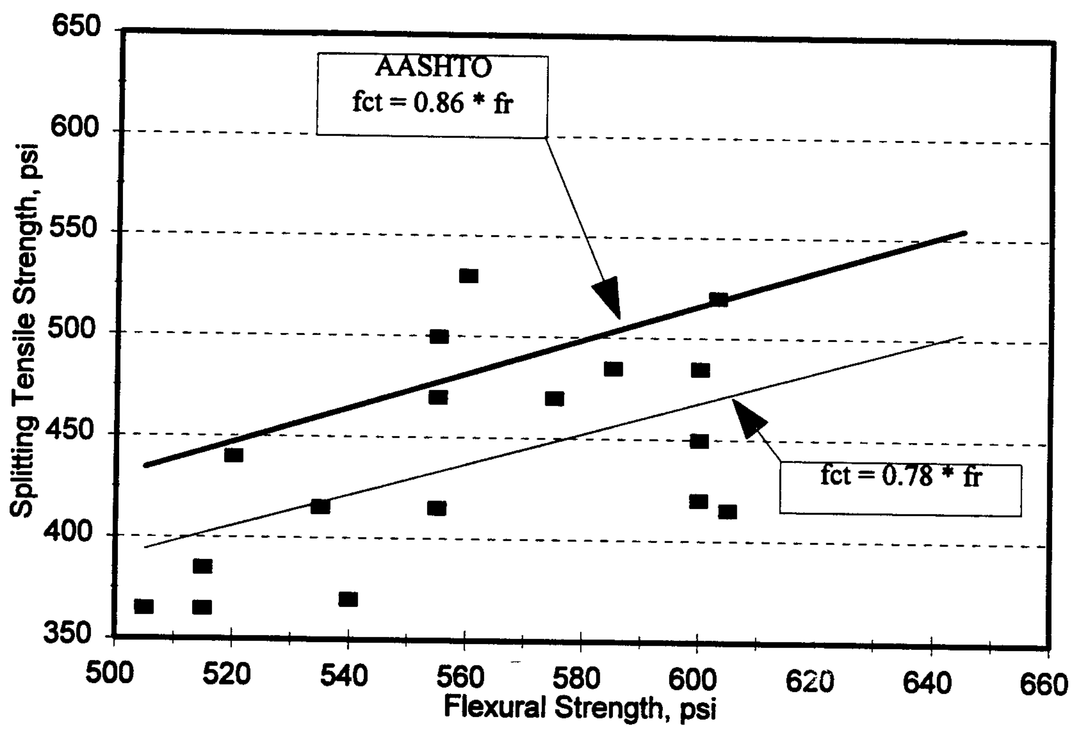


Figure 3.12B Plot of Flexural Strength vs. Experimental and Calculated Splitting Tensile Strengths, US Customary Units

4. INVESTIGATION OF INTERFACIAL SHEAR BOND STRENGTH

4.1 Introduction

In this chapter, the results of interfacial shear bond strength of recycled aggregate concrete to concrete with natural aggregates are presented. The recycled aggregate concrete consisted of 60%, by volume, of recycled coarse aggregate with varying volume percentage of recycled fine aggregate. Two different test methods for obtaining interfacial bond strength were used. These are the Double L Interfacial Shear Bond Strength Test developed at NCSU for the Strategic Highway Research Program (SHRP C-205) and the ASTM C882-91 Slant Shear Test.

The focus of the interfacial bond strength test was: *(i)* to study the effects of using recycled aggregates on interfacial shear bond strength, and *(ii)* to compare and evaluate the results from two notable interfacial bond test methods. These are the Double L Bond Test developed by the NCSU for the Strategic Highway Research Program, SHRP C-205 and the ASTM C882-91 Slant Shear Test.

For the Double L interfacial Shear Bond Strength Test and the ASTM Slant Shear Test, in order to mimic a condition of an older existing concrete pavement, the concrete specimens were considered to be mature after 28 days. The test age for the overlay concrete was seven days. All specimens portraying mature concrete were cast using the "NCDOT Mix" design. Specimens bonded to mature specimens were cast with concrete which had varying percentage of recycled fine aggregate. The content of recycled fine aggregate was varied from 0, 25, 50, 75, and 100 percent of the total volume of the fine aggregates in a mixture, while holding the volume of recycled coarse aggregate constant at 60 percent of the total coarse aggregate content. A control test of the "NCDOT Mix" bonded to a mature "NCDOT Mix" was also performed. The test matrix showing the number of specimens and test type is shown in **Table 4.1.**

4.2 Testing Procedure

4.2.1 Fabrication of Double L Bond Specimens

All specimens were prepared in accordance with *SHRP Product No. 2025* [12]. As shown in **Figure 4.1**, the concrete-to-concrete Double L bond specimens were composed of an inverted L-shaped segment "A" bonded to an upright L-shaped segment "B". The "NCDOT Mix" design, presented in **Table 2.8**, was used to cast one segment. This segment represented the concrete of a mature existing pavement, and is called segment "A". Concrete using recycled aggregate was used for the other segment of the Double L specimen. This segment termed as "B" represented the concrete of a pavement overlay. For segment "B", the volume of recycled coarse aggregate was held constant at 60% of the total volume of coarse aggregate. Six sets of segment "B"s were cast on top of their respective NCDOT segment "A"s. Each set consisted of two replicate specimens. Five sets of the overlay concretes varied in fine aggregate volume from 0, 25 ,50, 75, and 100% while the sixth set was a control specimen using the "NCDOT Mix" design. The mixture proportions for all concrete mixtures using different percentages of recycled aggregates as replacement of natural aggregates are shown in **Appendix A, Table A.2.**

Each segment of the Double L specimen had a height of 380 mm (15 in.) on the long side and 152 mm (6 in.) on the short side. The width of the base was 305 mm (12 in.) and the depth at all places is 152 mm (6 in.) as shown in **Figure 4.1**. This specimen is referred to as a 380 x 152 x 305 x 152 mm (15 x 6 x 12 x 6 inches) specimen and the nomenclature is L_l x L_s x B x D, where L stands for the length of both the long and short sides, B stands for width, and D represents depth. Each segment was reinforced with two #3 bars and two #2 bars to avoid premature cracking or failure in bending. Two 100 mm (4 in.) pieces of 9.5 mm (3/8 in.) threaded rod were inserted into the large end of the form to be used later for attaching steel end-plates. Lifting handles made from #2 smooth bars were installed on each of the segments. The forms were constructed from steel channels and plates which were held together by clamps. They were anchored to a plywood base as shown in **Figure 4.2** and **Figure 4.3.** The forms and base were lightly oiled before the concrete was placed. Three 100 x 200 mm (4 x 8 in.) companion cylinders were also cast along with each segment of the Double L specimens. These cylinders were used to determine the compressive strength and modulus of elasticity at the time of testing.

For casting the segment "B", an area of 23200 mm^2 (36 in^2) of the mature concrete was used as part of the formwork. This became the bond surface. The dimensions of the bonding surface were chosen so that the size effect of the aggregate at the bond area would be minimal. This bond surface area was large enough to give a representative strength of a field condition.

The specimens cast from the NCDOT mix were stripped and cured in a moist room with relative humidity of 100% for 28 days. The bond surface was then sand blasted by NCDOT and Parker Monuments using extra fine BX-30 sand blasting sand under 0.620 MPa (90 psi) nozzle pressure for a period of 15 to 20 seconds. This was to simulate the scarifying process used in the field in order to create a better bonding surface. When the concrete with recycled aggregate was placed, the specimen was turned on its side with the bond surface facing up. Prior to the placement of the concrete with recycled aggregate, the sandblasted surface of the mature concrete with the NCDOT mix was wetted for 30 minutes by spraying the contact surface with water. The setup for casting of the 380 x 152 x 305 x 152 mm (15 x 6 x 12 x 6 inches) overlay specimen is shown in **Figure 4.4.** The Double L specimen was then cured for three days with burlap and covered with plastic and wetted twice a day. On the third day the specimen was moved into the 100% humidity room and cured for four more days before testing.

4.2.2 Fabrication of ASTM C882-91 Slant Shear Specimen

The slant shear specimens consist of two segments. These two segments have a common interface at an angle of 30 with respect to the vertical. Concrete was cast into rigid plastic cylinder molds 76 mm (3 in.) in diameter and 152 mm (6 in.) in height in accordance with *ASTM C882-91, Standard Test Method for Bond Strength of Epoxy-Resin Systems Used With Concrete by Slant Shear* [5]. Even though this test is typically used with adhesives, the test can be modified for bonding concrete without adhesives [13]. For casting of the mature concrete, dummy molds were machined of solid PVC pipe to the dimensions shown in **Figure 4.5.** The dummy sections consisted of half the volume of the cylinder, but at an angle of 30 from the vertical. The finished dummy molds are shown in front of their respective cylinder molds in **Figure 4.3**. This dummy section was then placed in the bottom of the rigid plastic cylinder mold with the slant side facing up. The sides of the molds were lightly oiled along with the cylinder to facilitate the debonding of the dummy section. Concrete using the "NCDOT Mix" proportions was placed on top of the dummy section, vibrated and rodded with a 9.5 mm (3/8 in.) diameter tamping rod. The concrete was placed in three layers and finished even to the top of the cylinder mold. The finished specimen was covered with a plastic sheet to minimize the loss of moisture. These half sections were stripped one day after pouring, then placed in the moist room for at least 28 days.

The slanted face of the mature half specimens were then sandblasted using the same procedure used for the Double L bond specimens. The mature concrete half specimens were placed in the bottom of a lightly oiled cylinder mold and the slanted face was prewetted 30 minutes in advance of pouring new concrete. For casting the new concrete, the same procedure was used as that for casting the mature concrete against the dummy section. The whole slant shear specimen was stripped one day after pouring and placed in the moist room for 6 days to cure. A total of 3 replicate slant shear specimens were cast from each batch of concrete along with two Double L specimens.

4.2.3 Double L Interfacial Shear Bond Strength Testing

The Double L bond tests were conducted when the new concrete had reached the age of 7 days. A steel plate was attached to each end of the specimens, which was then set upright in the testing machine. The specimen was then placed on two more steel plates that interlocked with the first set. The plates aided in distributing the load evenly across the top and bottom faces with the resultant force in the plane of the bond surface.

For safety reasons, blocks of wood were inserted into the gap in between the upper and lower segments of the Double L specimen. The specimen was loaded at a rate of 6.7 kN/min (1500 lb/min). The test setup for the Double L specimen is shown in **Figure 4.6.**

4.2.4 ASTM C882-91 Slant Shear Testing

The slant shear bond tests were conducted when the new concrete had reached the age of 7 days. The testing procedure involved loading the specimen in compression. The test was conducted as a standard compression test using ASTM C 39 - *Test Method for Compressive Strength of Cylindrical Concrete Specimens* [5]. For this study, an unbonded capping system was used. It consisted of steel restraining rings with a neoprene pad insert on top and sand on bottom. Earlier unpublished work at NCSU had shown that use of sand in the bottom resulted in cylinder breaks with more vertical cracking at failure. The loading rate for a 76 x 152 mm (3 x 6 in.) cylinder was 66.7 kN/min. (15,000 lbs/min.). The test setup for the ASTM Slant Shear Specimen testing is shown in **Figure 4.7**.

4.3 Test Results and Discussion

4.3.1 Interfacial Bond Strength Test Results Using the Double L Bond Specimen

For this test, specimens were loaded until the interface debonded. The load corresponding to this is termed as failure load or debonding load. **Table 4.2** shows the results of the interfacial bond strength testing. **Table 4.3** gives the results of the compressive strength tests performed on the companion cylinders. **Figure 4.8** depicts the effects of the percentage of recycled fine aggregate on the interfacial bond strength of the concrete. The results indicate a smooth decrease in interfacial bond strength with increasing percentage of recycled fine aggregate. The control specimen for this test was W6-NCDOT, with an age of 71 days at testing, bonded to W13-NCDOT, with an age of 7 days at testing. Both specimens W13-NCDOT and W6-NCDOT use the "NCDOT Mix" design.

Interfacial bond strength is limited by the direct tensile strength of the weaker of the two materials bonded together. In this investigation, the overlay, or the concrete utilizing recycled aggregate was the weaker material since the Double L bond specimens were tested at seven days after casting of the overlay concrete on the mature concrete segment of the specimen. Uniaxial tensile strength can be estimated by using the expression $0.54\ f'_c$ for SI units ($6.5 f'_c$ for U.S. Customary units) [14]. For all specimens except the specimen with 100% recycled fine aggregate, the estimated tensile strength values are lower than the interfacial bond strength values calculated from failure loads obtained from the interfacial bond test. Adequate interfacial bond strength was developed between the concretes with natural aggregates and concretes with 60% recycled coarse aggregates and various percentages of recycled fine aggregates. This implies that in an actual overlay pavement, failure will be governed by the tensile strength of the recycled aggregate concrete. It should be pointed out that tensile strength of RCA overlay is more critical than bond strength failure between RCA overlay and layer of concrete with natural aggregates.

The results for bond strength appear to be plausible and there is a trend of decreasing bond strength with increasing percentage of recycled fine aggregate. However, the difference in reduction of interfacial bond strength is only about 6% when the percentage of recycled fine aggregate is increased from 0% to 50%. The data also shows that interfacial bond strength of natural aggregate concrete to recycled aggregate concrete (60% RCA and 0% RFA 50%) is greater than that of the NCDOT control specimens (i.e. natural aggregate concrete to natural aggregate concrete). This can be attributed to the higher cement content in the concrete mixtures utilizing recycled aggregates.

4.3.2 Test Results from the ASTM C882-91 Slant Shear Test

Three specimens were cast for each series of bond tests for the concrete mixtures utilizing recycled aggregate and the control mixtures using natural aggregates. Specimens were loaded until failure was achieved. The failure load was recorded and according to the test procedure should have been divided by the slant face area of 9116 mm^2 (14.13 $in.^2$), subtracting from it any voids in the bonded surface. However, over half the slant shear specimens failed in compression mode as shown in **Figure 4.9** and the others failed partially along the inclined bond plane as shown in **Figures 4.10**. This photograph clearly shows that only part of the bonded plane failed and it was not easy to define the surface area that debonded or experienced sliding shear. Due to difficulties associated with identifying the area of failure surface, the bond strength could not be computed. **Table 4.4** gives the failure load results from the slant shear test for all batches.

From the results (**Table 4.3**), it appears that the slant shear test may not be a satisfactory test procedure to measure interfacial bond strength. The slant shear specimens did not fail in the intended mode. Even if the specimens had failed along the slanted bond face, the computed bond strength, using the smallest load in the data, would have been approximately 5.03 MPa (730 psi) for W15-60-100 which is 2.5 times greater than the interfacial bond strength obtained using the Double L interfacial bond strength test. This further indicates that the slant shear test may not be a representative test for estimating the interfacial bond strength of concrete.

Table 4.1. Test Matrix for Interfacial Bond Strength Using the Double L Bond Test and the ASTM Slant Shear Test.

Batch ID of Mature Concrete	Compressive Cylinders for Mature Concrete	Double L Bond Test	ASTM Slant Shear Test	Batch ID of Overlay Concrete	Compressive Cylinders for Overlay Concrete
W2-NCDOT	3	2	3	*W8-60-0*	3
W3-NCDOT	3	2	3	*W9-60-25*	3
W4-NCDOT	3	2	3	*W10-60-50*	3
W5-NCDOT	3	2	3	*W12-60-75*	3
W11-NCDOT	3	2	3	*W15-60-100*	3
W6-NCDOT	3	2	3	*W13-NCDOT*	3

Notes:
The Batch ID follows the format in this example.
W8-60-0
where: W8 - designates the batch number
60 - designates the volumetric percentage of recycled coarse aggregate (RCA)
0 - designates the volumetric percentage of recycled fine aggregate (RFA)

Table 4.2. Results of Interfacial Bond Strength Tests Using Double L Specimens

Batch ID of Mature Concrete (Segment A)	Test Age, Days	Batch ID of Overlay Concrete (Segment B)	% of RFA	Test Age, Days	Double L Interfacial Bond Strength, MPa (psi)		
					Test 1	Test 2	Average
W2-NCDOT	**37**	*W8-60-0*	0	**7**	2.80 (406)	3.02 (439)	2.91 (422)
W3-NCDOT	**38**	*W9-60-25*	25	**7**	2.79 (405)	2.87 (416)	2.83 (411)
W4-NCDOT	**42**	*W10-60-50*	50	**7**	3.16 (458)	2.32 (336)	2.74 (397)
W5-NCDOT	**60**	*W12-60-75*	75	**7**	2.43 (352)	2.46 (357)	2.45 (355)
W11-NCDOT	**36**	*W15-60-100*	100	**7**	2.88 (418)	1.15 (167)	2.02 (293)
W6-NCDOT	**71**	*W13-NCDOT*	0	**7**	2.39 (347)	2.53 (368)	2.46 (357)

Table 4.3. Results of Companion Compressive Strength Tests

Batch ID of Mature Concrete	**Test Age, Days**	**Average Compressive Strength, MPa (psi)**	**Batch ID of Overlay Concrete**	**Test Age, Days**	**Average Compressive Strength, MPa (psi)**
W2-NCDOT	37	34.9 (5060)	W8-60-0	7	22.1 (3210)
W3-NCDOT	38	40.6 (5890)	W9-60-25	7	22.2 (3220)
W4-NCDOT	42	40.1 (5810)	W10-60-50	7	22.1 (3200)
W5-NCDOT	60	45.5 (6600)	W12-60-75	7	19.4 (2810)
W11-NCDOT	36	35.9 (5210)	W15-60-100	7	18.0 (2610)
W6-NCDOT	71	40.0 (5810)	W13-NCDOT	7	25.3 (3660)

Table 4.4. Test Results from ASTM Slant Shear Tests

Batch ID of Mature Concrete (Segment A)	**Test Age, Days**	**Batch ID of Overlay Concrete (Segment B)**	**% of RFA**	**Test Age, Days**	**ASTM Slant Shear Load, kN (kips)**			
					Test 1	**Test 2**	**Test 3**	**Average**
W2-NCDOT	**37**	*W8-60-0*	0	**7**	69 (15.5)	68 (15.3)	71 (16.0)	69 (15.6)
W3-NCDOT	**38**	*W9-60-25*	25	**7**	54 (12.1)	74 (16.7)	76 (17.0)	68 (15.3)
W4-NCDOT	**42**	*W10-60-50*	50	**7**	62 (13.9)	67 (15.1)	67 (15.1)	65 (14.7)
W5-NCDOT	**60**	*W12-60-75*	75	**7**	48 (10.9)	65 (14.7)	66 (14.9)	60 (13.5)
W11-NCDOT	**36**	*W15-60-100*	100	**7**	46 (10.3)	49 (11.0)	60 (13.5)	52 (11.6)
W6-NCDOT	**71**	*W13-NCDOT*	0	**7**	70 (15.7)	72 (16.1)	75 (16.8)	72 (16.2)

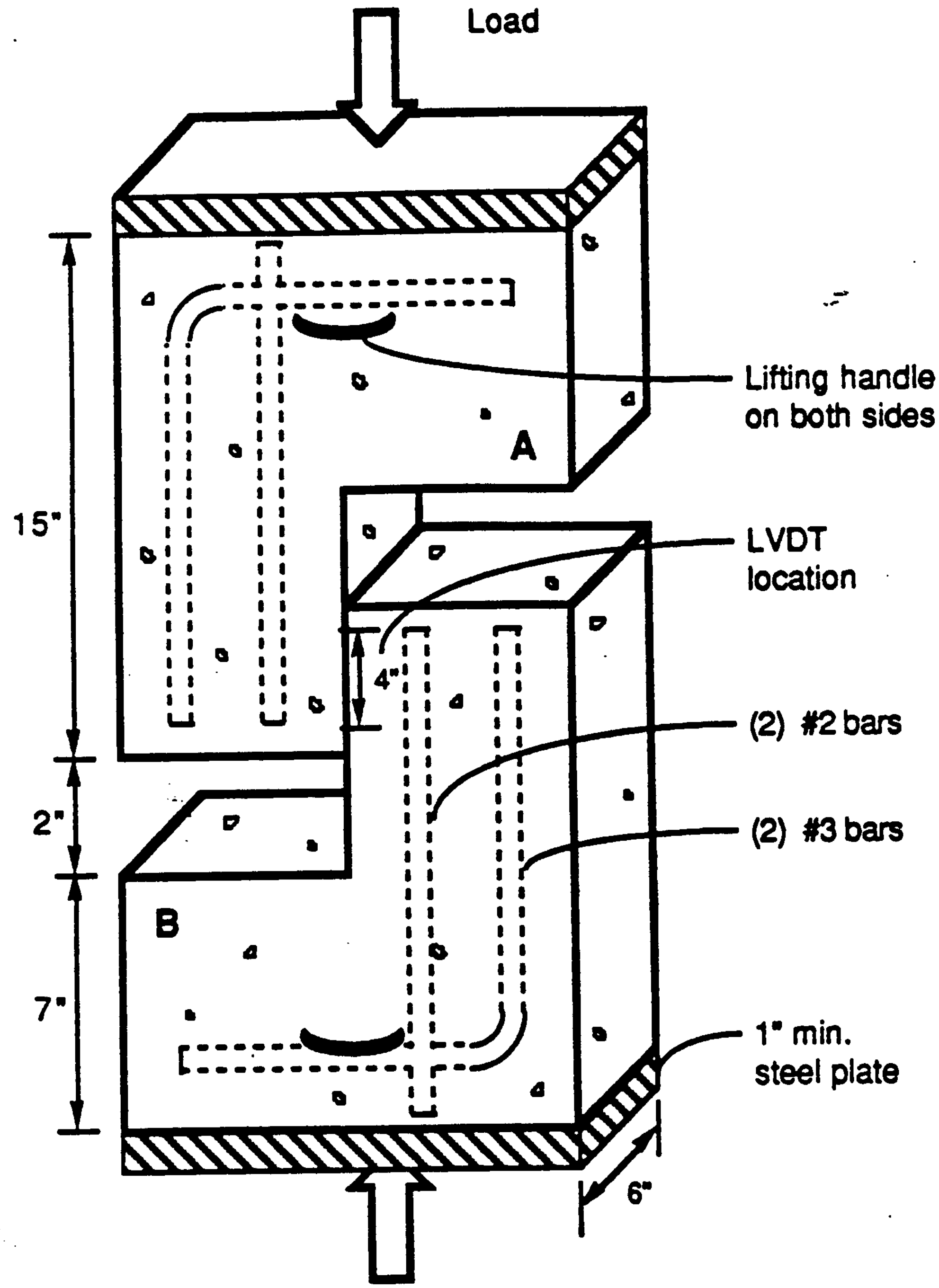


Figure 4.1. Dimensions and Steel Layout for the Double L Bond Specimen [12]

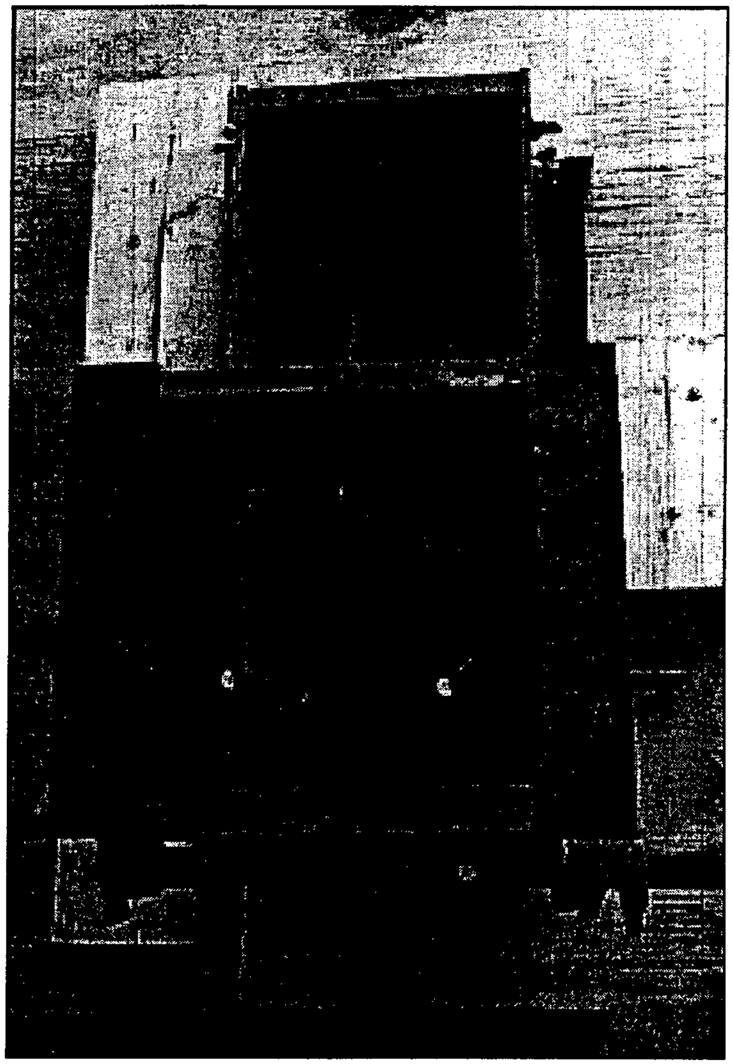


Figure 4.2. Top View of the Steel and Formwork for the Double L Bond Specimen

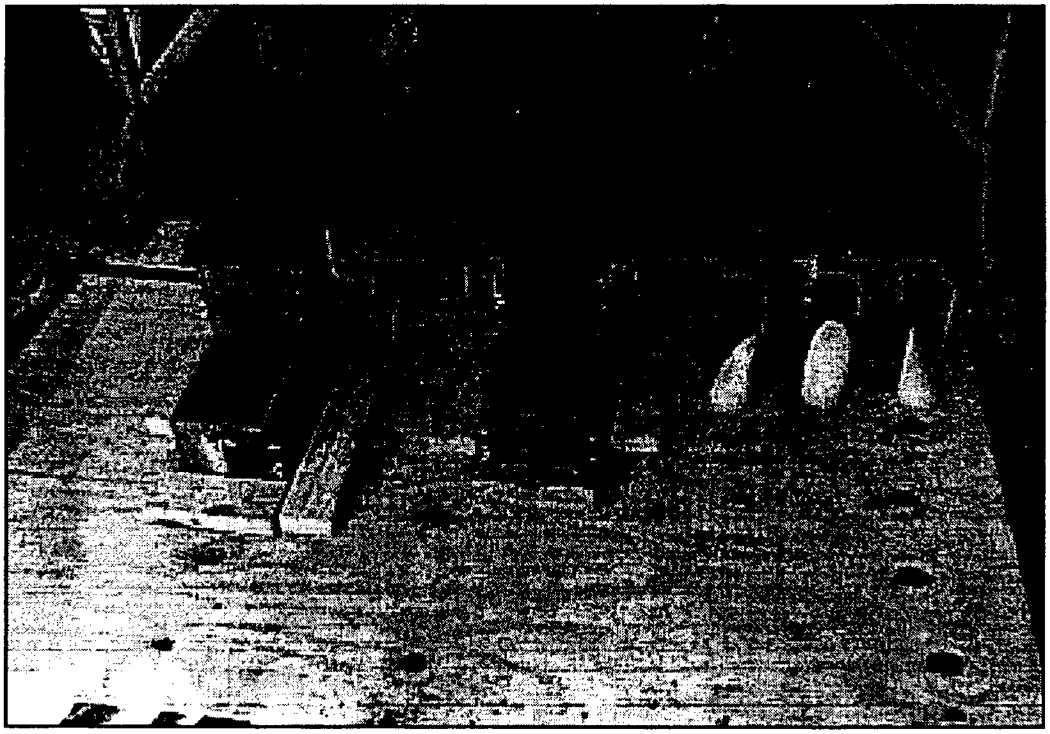


Figure 4.3. Setup for Casting of Segment "A" of the Double L Bond Specimen and Finished Dummy Molds for Slant Shear Specimens

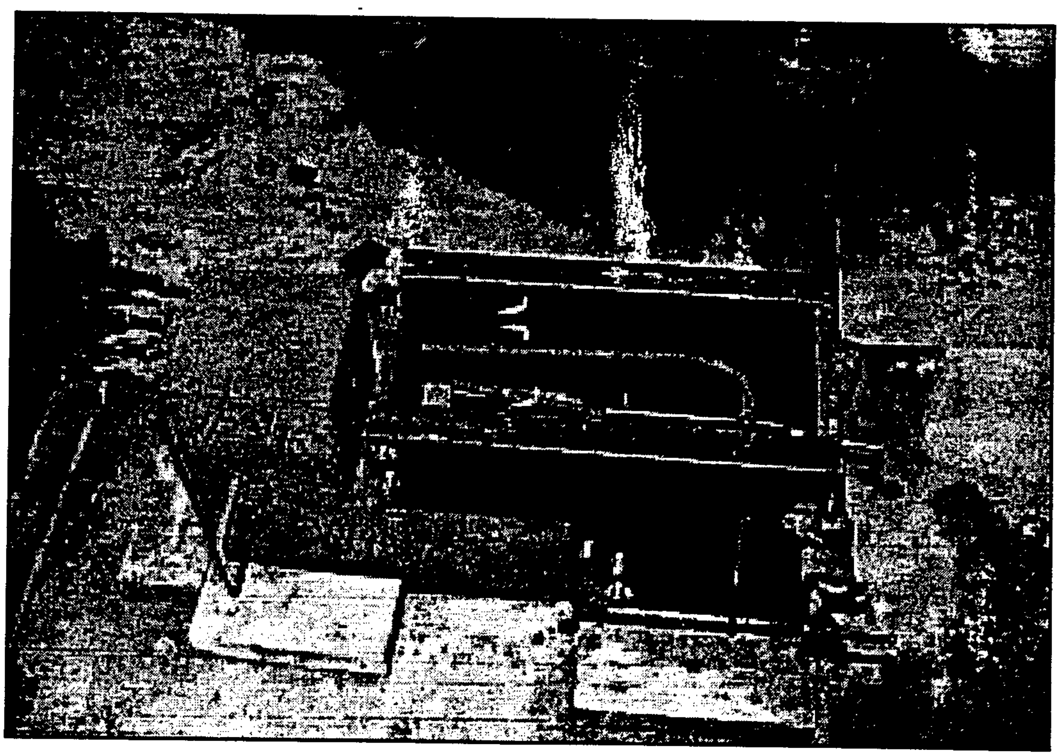
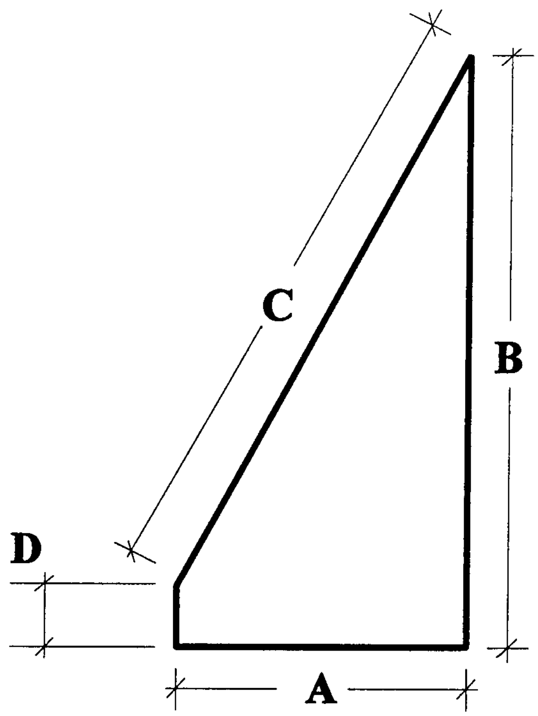


Figure 4.4. Setup for Casting Segment "B" of the Double L Bond Specimen



A = 76.2 mm (3.000 in.)

C= 152.4 mm (6.000 in.)

B = 142.2 mm (5.598 in.)

D= 10.2 mm (0.402 in.)

Figure 4.5. Dimensions of Plastic Dummy Mold Cylinder Used in Casting ASTM Slant Shear Specimens

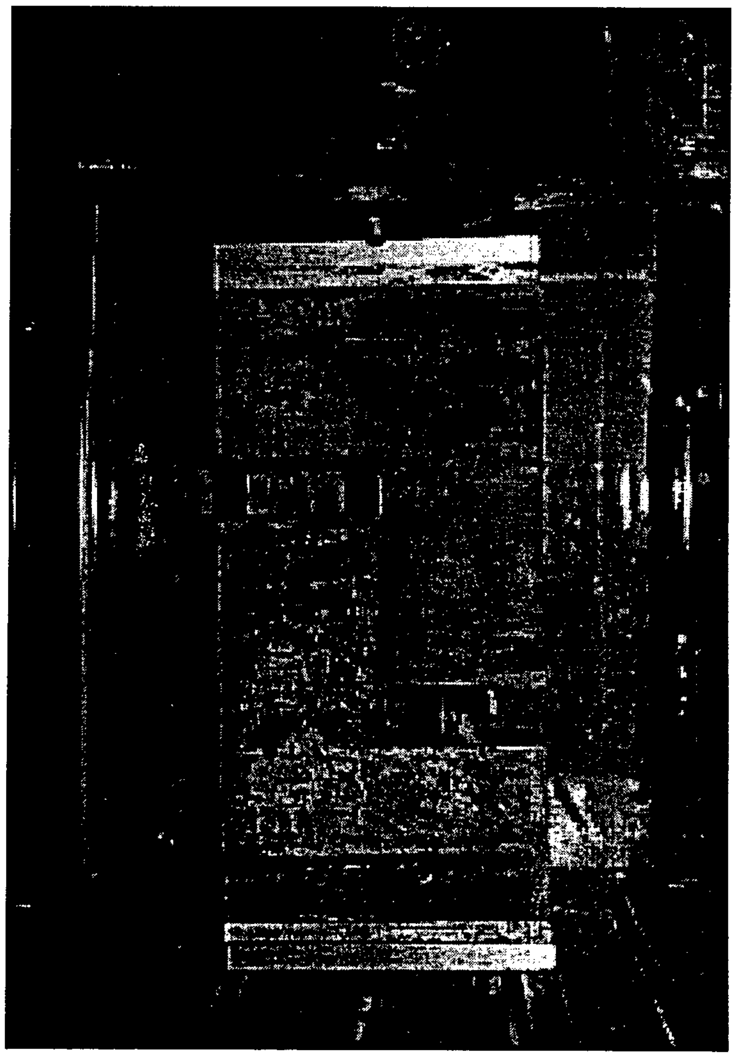


Figure 4.6. Testing for Interfacial Bond Strength Using the Double L Bond Specimen

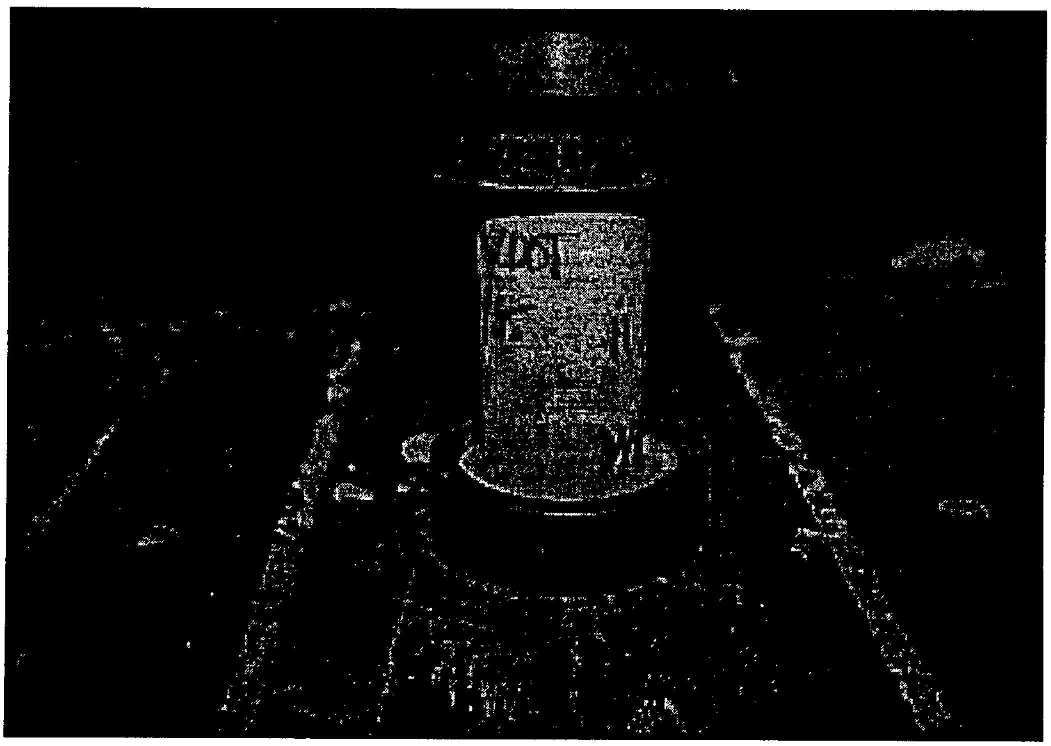


Figure 4.7. Testing for Bond Strength Using ASTM C882-91 Slant Shear Test

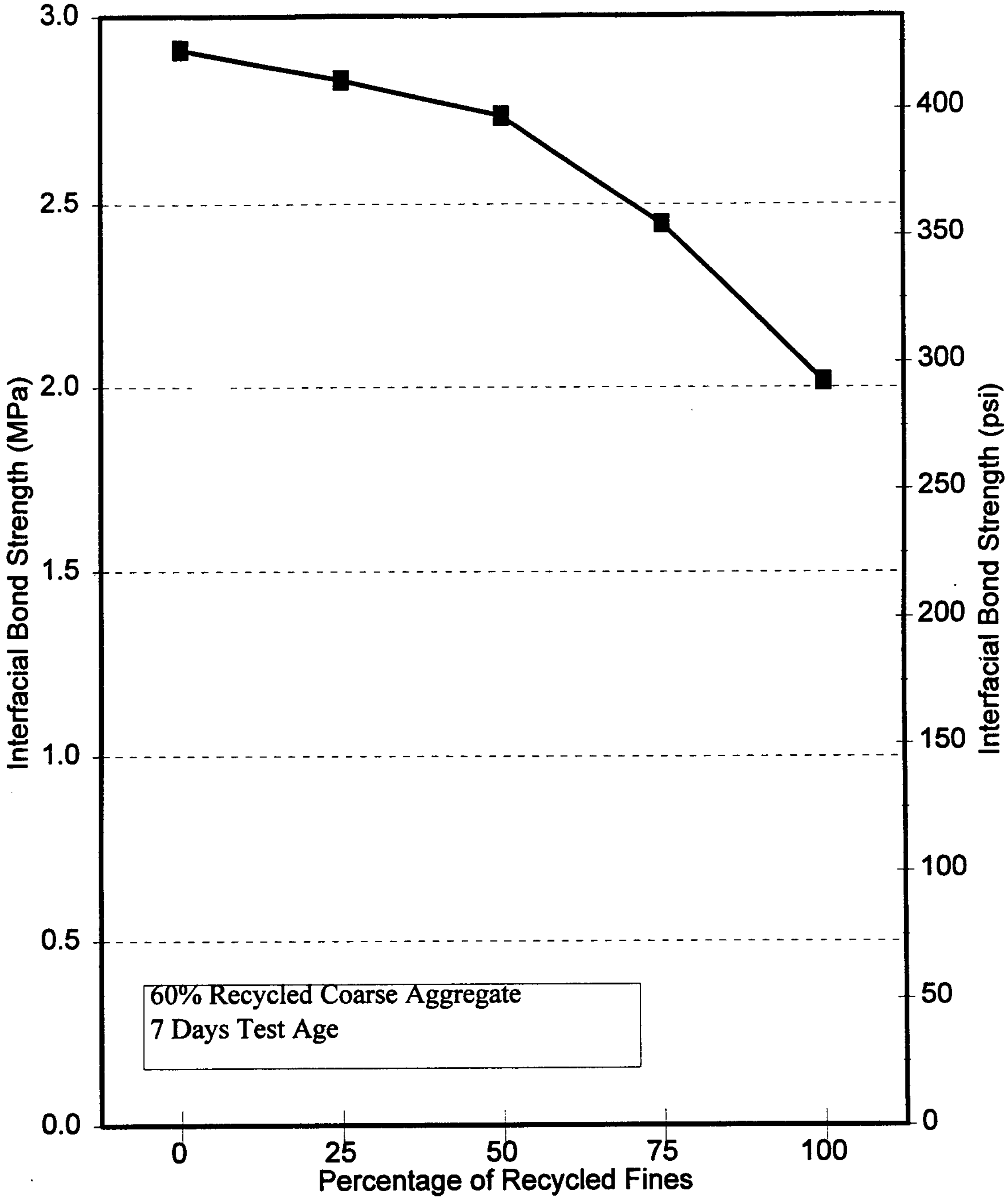


Figure 4.8. Interfacial Bond Strength vs. Percentage Recycled Fine Aggregate for the Double L Bond Specimen

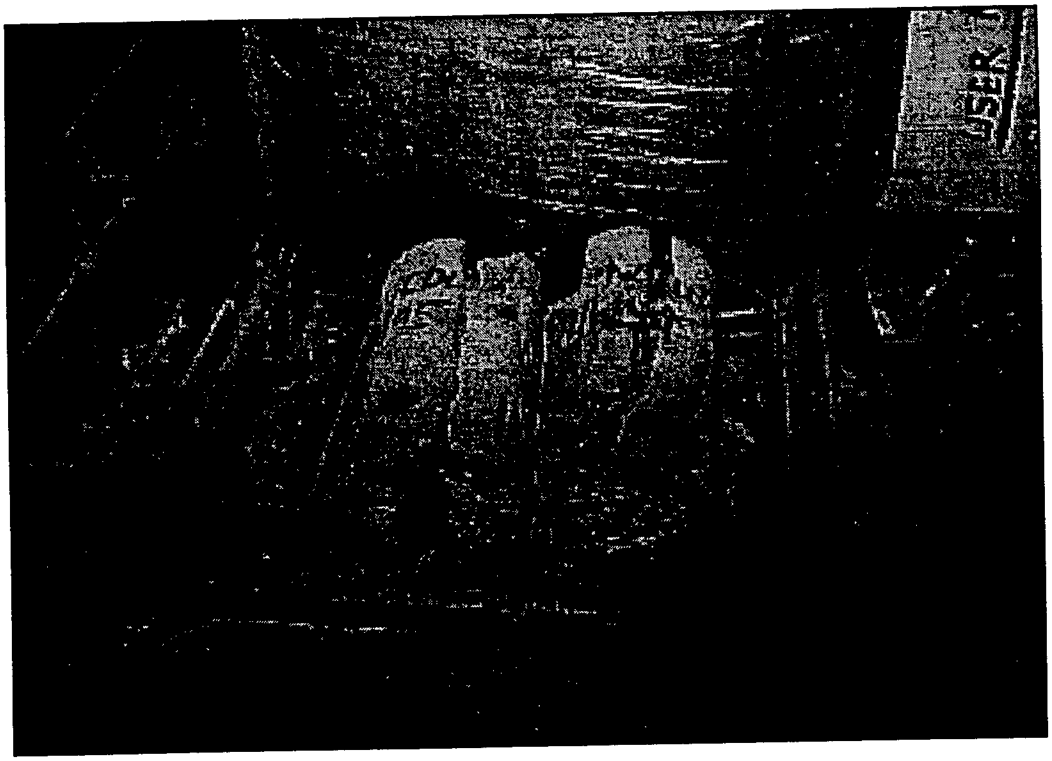


Figure 4.9. ASTM Slant Shear Specimens After Failure in Compressive Mode

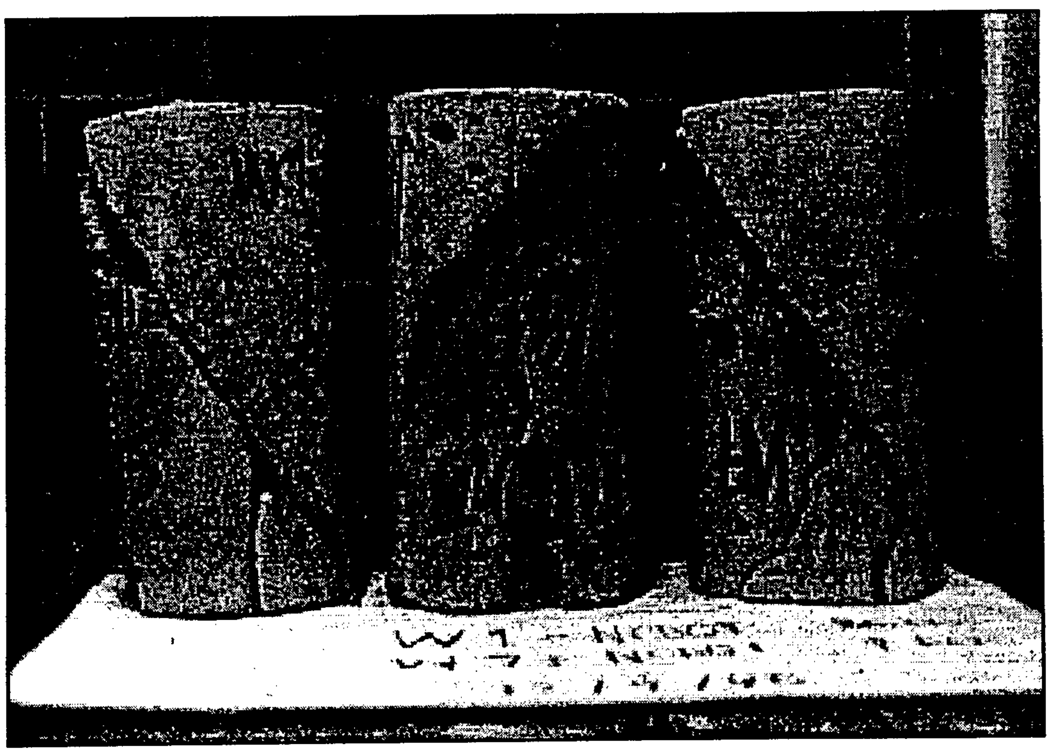


Figure 4.10. Profile of Slant Shear Specimens After Failure

5. IMPLEMENTATION AND TECHNOLOGY TRANSFER

The effects of recycled coarse and fine aggregates on the properties of concrete were investigated. The results indicate that using 100% recycled coarse aggregate produces acceptable quality control. Concretes with 60% RCA and up to 30% RFA produce concretes which can be used for variety of pavement applications.

Concretes with recycled coarse aggregates can be used in concrete shoulders, median barriers and base course of the highway pavements. The fine aggregates from the crushing operation can be used for subgrade corrections.

Implementation of the recommendations can be accomplished with minor modifications of the relevant specifications.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions - Mechanical Properties

Based on the results of this study, using the materials and ranges of variables given, the following conclusions can be drawn for properties of fresh and hardened concretes made with recycled coarse and fine aggregates:

1. Increasing the percentage of recycled fine aggregate resulted in a loss of slump, and hence workability. This could be corrected by addition of a water reducer to bring the slump back within the target range of 50 mm ± 25 mm (2 in. ± 1 in.).
2. As the percentage of recycled fine aggregate increased, the air content of the fresh concrete decreased. To meet the target air content of 5.5% ± 1.5%, this decrease was corrected by increasing the dosage of air entraining agent. For the concrete mixture utilizing all recycled aggregates it was not possible to produce a mixture to satisfy the target range for air content.
3. Increasing the percentage of recycled aggregate resulted in the concrete having a lower unit weight. This was expected due to the fact that the recycled aggregates had lower specific gravities than the natural aggregates.
4. For hardened concrete, the compressive strength, elastic modulus, flexural strength, and splitting tensile strength showed no clear trend with the substitution of recycled coarse aggregate in the range considered in this study. These properties did show an overall decrease with the substitution of recycled fine aggregate. The relative magnitude of this decrease was independent of the amount of recycled coarse aggregate used in the mixture.
5. The compressive strength at the test age of 28 days for concretes made with 100% recycled fine aggregate was 25% to 30% lower than comparable concretes made with 100% natural fine aggregate.
6. The elastic modulus at the test age of 28 days for concretes made with 100% recycled fine aggregate was 28% to 40% lower than comparable concretes made with 100% natural fine aggregate.

7. The flexural strength at the test age of 28 days for concretes made with 100% recycled fine aggregate was 15% to 20% lower than comparable concretes made with 100% natural fine aggregate.
8. The splitting tensile strength at the test age of 28 days for concretes made with 100% recycled fine aggregate was 18% to 27% lower than comparable concretes made with 100% natural fine aggregate.
9. Based on the variability and range of the experimental data, the empirical equation recommended by AASHTO and ACI 318 for estimating elastic modulus seems to be applicable for recycled aggregate concretes.
10. The empirical relationships recommended by ACI 330 and ACI 325 for estimating the flexural strength as a function of compressive strength are unconservative for recycled aggregate concretes. The relationship recommended in ACI 318-95 represented a lower bound estimate of the experimental data.
11. The ACI 318-95 equation for splitting tensile strength is unconservative for recycled aggregate concretes. The lower bound of the experimental data could be represented by $f_{ct} = 0.54\ \sqrt{f_c}$ for SI units and $f_{ct} = 6.5\ \sqrt{f_c}$ for US Customary units. The AASHTO recommendation for estimating the splitting tensile strength as 86% of the flexural strength was found to unconservative for recycled aggregate concretes. The experimental data showed that for concretes with recycled aggregates, the average splitting tensile strength could be estimated as 78% of the flexural strength.

Based on the results of this study, it can be determined that recycled coarse aggregate can be used as a substitute for natural coarse aggregates. However, recycled fine aggregate is not acceptable as a complete substitute for natural fine aggregate. It may, however, be suitable as a partial replacement for natural fine aggregate. Most departments of transportation specify either no recycled fine aggregate, or less than 30% replacement of natural sands with recycled fines. A similar specification that is often used is to allow up to 25% replacement of natural sands with manufactured sands. Manufactured sands, like recycled fine aggregates, are very harsh and difficult to finish.

The NCDOT requirement of a minimum flexural strength of 3.79 MPa (550 psi) at 14 days was met by the mixtures with 60% recycled coarse aggregate mixtures, as long as the volumetric percentage of recycled fine aggregate was less than 50 percent. Considering the field conditions and the imposing a margin of safety, it is recommended that mixtures with 60% RCA and up to 30% RFA can be produced to meet the minimum flexural strength requirements for pavement applications.

6.2 Conclusions - Bond Testing

Based on the results, the following conclusions can be drawn regarding the interfacial bond strength of concrete mixtures utilizing recycled aggregates with concretes using natural aggregates. These conclusions are for concrete with 60% recycled coarse aggregate (RCA) by volume.

1. The Double L Interfacial Bond Strength Test is a reliable test for obtaining interfacial bond strength values. Results from these tests were very consistent for replicate specimens. For concretes with recycled aggregate, interfacial bond strength decreases nonlinearly with increasing volume percentage of recycled fine aggregate.
2. Interfacial bond strength of recycled aggregate concretes (60% RCA and 0% RFA 50%) to natural aggregate concrete was higher than that for NCDOT mixtures using natural aggregates. This can be attributed to the increased cement content used for concrete mixtures with recycled aggregates.
3. Interfacial bond strengths computed from the interfacial bond test were higher than the estimated tensile strength of the weaker concrete (i.e. concrete with recycled aggregates). This shows that adequate interfacial bond strength was developed and failure in this type of overlay pavement will most likely be governed by the tensile strength of the recycled aggregate concrete.
4. For the ASTM C882-91 Slant Shear Test, over three quarters of the specimens did not fail in the intended mode when tested at the design age of 7 days for the overlay. If specimens had failed along the inclined bond plane, the computed bond strength would have been 2.5 time greater than that for the Double L interfacial bond test.

6.3 Recommendations for Further Work

This study investigated the effects of recycled aggregates on the mechanical properties and interfacial bond strength of concretes. However, there are other concerns with recycled aggregates that must be investigated. The effects of recycled aggregates on freeze-thaw resistance, fatigue strength, creep, and shrinkage must also determined. Moreover, the recycled aggregate that was used in this study came from only one source. To have a better understanding of how recycled aggregate affects the properties of concrete, aggregates from different concrete pavements must be investigated. Furthermore, to be able to economically use any recycled fine aggregates in concrete mixtures, the difficulty in determination of water absorption must be eliminated. A standardized substitute for the current ASTM C128 procedure must be developed.

Another aspect of research that must be investigated is the variability of the recycled aggregates. Mixtures made with the recycled fines had high variation in the quality assurance tests for slump and air content. In addition, the variation in strength characteristics must be better understood. An increased comprehension of the effect of recycled aggregates on the reliability factor and standard deviation would also be beneficial.

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APPENDICES

APPENDIX A

This appendix includes the target batch proportions for all the batches in this study. **Table A.1** details the batch proportions for the "NCDOT Mix" and the "NCSU Mix 1" control mixture. The amount of aggregates used in the recycled batches are presented in **Table A.2**. For these batches, the amounts of cement and water are the same as for the "NCSU Mix 1" presented in **Table A.1**. Target amounts of AEA and HRWR are not indicated since these changed depending on the actual material quantities used for of each batch. The amounts of AEA and HRWR actually used for each batch are shown in **Appendix B, Table B.1**.

The Mix ID nomenclature for **Table A.2** follows the format in this example:

B-60-25

where: 60 - Designates the volumetric percentage of recycled coarse aggregate of the total coarse aggregate

25 - Designates the volumetric percentage of recycled fine aggregate of the total fine aggregate

Table A.1. Target Mixture Proportions for Concretes with Natural Aggregates

Materials	Quantity	
	NCDOT Mix	NCSU Mix 1
Cement, kg/m^3 (pcy)	312 (526)	368 (620)
Coarse Aggregate, kg/m^3 (pcy)	1112 (1875)	1079 (1819)
Fine Aggregate, kg/m^3 (pcy)	722 (1217)	647 (1091)
Water, kg/m^3 (pcy)	148 (250)	173 (291)
AEA, L/m^3 (oz/yd^3)	0.17 (4.5)	0.17 (4.5)
HRWR, L/m^3 (oz/yd^3)	1.20 (31)	3.48 (90)

Table A.2. Target Aggregate Proportions for Concrete Mixtures with Recycled Aggregates

Mix ID	Natural Coarse Aggregate kg/m^3 (pcy)	Recycled Coarse Aggregate kg/m^3 (pcy)	Natural Fine Aggregate kg/m^3 (pcy)	Recycled Fine Aggregate kg/m^3 (pcy)
B-60-0	432 (782)	603 (1016)	647 (1091)	0
B-60-25	432 (782)	603 (1016)	485 (818)	151 (254)
B-60-50	432 (782)	603 (1016)	323 (545)	301 (507)
B-60-75	432 (782)	603 (1016)	162 (273)	452 (761)
B-60-100	432 (782)	603 (1016)	0	602 (1014)
B-100-0	0	1006 (1695)	647 (1091)	0
B-100-25	0	1006 (1695)	485 (818)	151 (254)
B-100-50	0	1006 (1695)	323 (545)	301 (507)
B-100-75	0	1006 (1695)	162 (273)	452 (761)
B-100-100	0	1006 (1695)	0	602 (1014)

Notes:

1. All mixtures with recycled aggregates contained 368 kg/m^3 (620 pcy) of cement, and were made at a constant w/c ratio of 0.47. The amount of AEA and HRWR was varied to meet the target ranges of slump and air content.

APPENDIX B

A total of 50 batches were proportioned and mixed for this study. Many of these batches failed to meet the target slump and air content requirements. The seventeen mixtures which met both slump and air content requirements are presented in this appendix. To obtain a field usable mixture, the amounts of AEA and HRWR were adjusted to obtain a target air content between 5.5% ± 1.5%, and a target slump between 50 mm ± 25 mm (2 in. ± 1 in.). The actual amounts of AEA and HRWR used in each batch are shown in **Table B.1**. The results of tests conducted on the fresh concrete of each batch are presented in **Table B.2**.

The mix ID nomenclature follows the format in this example:

B37-60-25

where: B37 - Designates the batch number for record keeping
60 - Designates the volumetric percentage of recycled coarse aggregate
25 - Designates the volumetric percentage of recycled fine aggregate

Additionally, the batches using the NCDOT's original mixture design are designated B4-NCDOT, where B4 is the batch number.

Table B.1. Actual AEA and HRWR Amounts Used in Each Batch

Mix ID	Cement Content kg/m^3 (pcy)	AEA		HRWR	
		L/m^3	oz/yd^3(oz/cwt)	L/m^3	oz/yd^3 (oz/cwt)
B4-NCDOT	312 (526)	0.17	4.5 (0.86)	1.2	30.9 (5.9)
B-0-0	368 (620)	0.17	4.5 (0.73)	3.26	84.3 (13.6)
B20-60-0		0.38	9.7 (1.6)	1.88	48.7 (7.9)
B31-60-0		0.43	11.0 (1.8)	0.28	7.3 (1.2)
B21-60-25		0.71	18.3 (2.9)	1.88	48.7 (7.9)
B37-60-25		0.59	15.2 (2.5)	0.47	12.2 (2.0)
B22-60-50		0.94	24.3 (3.9)	2.35	60.9 (9.8)
B49-60-50	368 (620)	1.13	29.2 (4.7)	1.13	29.2 (4.7)
B23-60-75		1.41	36.5 (5.9)	2.35	60.9 (9.8)
B35-60-75		1.76	45.6 (7.4)	1.76	45.6 (7.4)
B24-60-100		1.88	48.7 (7.9)	2.94	76.1 (12.3)
B36-60-100		2.12	54.8 (8.8)	1.76	45.6 (7.4)
B44-60-100		0.52	13.4 (2.2)	0.45	11.6 (1.9)
B6-100-0		0.20	5.1 (0.82)	1.22	31.6 (5.1)
B11-100-25	368 (620)	0.25	6.4 (1.0)	0.88	22.8 (3.7)
B12-100-50		0.25	6.4 (1.0)	0.47	12.2 (2.0)
B14-100-75		0.98	25.3 (4.1)	0.71	18.3 (3.0)

Notes:
A total of 50 batches were proportioned and mixed. The 17 batches shown here met the target slump (50 mm ± 25 mm) and air content (5.5% ± 1.5%) requirements.

Table B.2. Fresh Concrete Test Results

Mix ID	Slump*		Air Content ** (%)	Unit Weight	
	mm	in		kg/m3	pcf
B4-NCDOT	38	1 1/2	4	2326	145.2
B-0-0	50	2	6.3	2268	141.6
B20-60-0	78	3 1/8	3.2	2352	146.8
B31-60-0	63	2 1/2	6.0	2236	139.6
B21-60-25	50	2	4.0	2300	143.6
B37-60-25	75	3	6.8	2268	141.6
B22-60-50	37	1 1/2	3.8	2287	142.8
B50-60-50	50	2	5.6	2211	138.0
B23-60-75	56	2 1/4	4.0	2255	140.8
B35-60-75	75	3	6.0	2204	137.6
B24-60-100	50	2	4.3	2217	138.4
B36-60-100	56	2 1/4	7.0	2159	134.8
B44-60-100	75	3	6.6	2140	133.6
B6-100-0	114	4 1/2	6.2	2236	139.6
B11-100-25	100	4	6.0	2198	137.2
B12-100-50	63	2 1/2	4.4	2236	139.6
B14-100-75	44	1 3/4	4.5	2211	138.0

* Specified slump = 50mm ± 25 mm.

** Specified air content = 5.5% ± 1.5%.

APPENDIX C

Data from the testing of hardened concrete specimens is detailed in this appendix. The twelve tables included are:

Table C.1. Test Results for Compressive Strength at 14 Days
Table C.2. Test Results for Compressive Strength at 28 Days
Table C.3. Test Results for Compressive Strength at 9 Months
Table C.4. Test Results for Modulus of Elasticity at 14 Days
Table C.5. Test Results for Modulus of Elasticity at 28 Days
Table C.6. Test Results for Modulus of Elasticity at 9 Months
Table C.7. Test Results for Flexural Strength at 14 Days
Table C.8. Test Results for Flexural Strength at 28 Days
Table C.9. Test Results for Flexural Strength at 9 Months
Table C.10. Test Results for Splitting Tensile Strength at 14 Days
Table C.11. Test Results for Splitting Tensile Strength at 28 Days
Table C.12. Test Results for Splitting Tensile Strength at 9 Months

Table C.1. Test Results for Compressive Strength at 14 Days

Mix ID	SI Units (MPa)				US Customary Units (psi)			
	Test 1	Test 2	Test 3	Avg.	Test 1	Test 2	Test 3	Avg.
NCDOT	30.3	29.2	33.9	31.1	4400	4230	4910	4510
NCSU Mix 1	50.1	52.2	50.7	51.0	7260	7580	7350	7400
B-60-0	32.4	30.7	32.1	31.7	4700	4450	4650	4600
B-60-25	27.8	29.2	29.0	28.7	4030	4230	4210	4160
B-60-50	28.9	27.7	28.4	28.3	4190	4020	4120	4110
B-60-75	26.3	26.3	26.3	26.3	3810	3820	3810	3810
B-60-100	22.1	21.9	22.0	22.0	3210	3170	3190	3190
B-100-0	32.3	30.8	31.2	31.4	4690	4470	4520	4560
B-100-25	27.8	30.1	29.9	29.3	4030	4370	4340	4250
B-100-50	27.6	29.4	29.4	28.8	4010	4260	4260	4180
B-100-75	28.5	27.6	28.3	28.1	4140	4000	4100	4080
B-100-100	---	---	---	---	---	---	---	---

Table C.2. Test Results for Compressive Strength at 28 Days

Mix ID	SI Units (MPa)				US Customary Units (psi)			
	Test 1	Test 2	Test 3	Avg.	Test 1	Test 2	Test 3	Avg.
NCDOT	38.4	29.7	34.4	34.2	5570	4310	4990	4960
NCSU Mix 1	49.6	56.5	53.5	53.2	7200	8190	7770	7720
B-60-0	36.3	33.9	37.4	35.9	5260	4920	5420	5200
B-60-25	31.9	31.6	31.2	31.6	4620	4590	4520	4580
B-60-50	30.2	29.9	30.1	30.1	4380	4330	4370	4360
B-60-75	27.6	28.0	28.2	27.9	4010	4060	4090	4050
B-60-100	27.2	26.8	24.6	26.2	3950	3890	3570	3800
B-100-0	33.1	37.6	35.9	35.5	4800	5460	5200	5150
B-100-25	---	---	---	---	---	---	---	---
B-100-50	---	---	---	---	---	---	---	---
B-100-75	---	---	---	---	---	---	---	---
B-100-100	---	---	---	---	---	---	---	---

Table C.3. Test Results for Compressive Strength at 9 Months

Mix ID	SI Units (MPa)				US Customary Units (psi)			
	Test 1	Test 2	Test 3	Avg.	Test 1	Test 2	Test 3	Avg.
NCDOT	46.4	45.9	47.7	46.7	6730	6650	6920	6770
NCSU Mix 1	64.6	65.8	63.6	64.7	9370	9550	9220	9380
B-60-0	52.3	56.5	56.8	55.2	7590	8200	8240	8010
B-60-25	46.7	52.4	49.4	49.5	6780	7600	7170	7180
B-60-50	47.1	47.4	41.5	45.3	6830	6880	6020	6580
B-60-75	44.8	42.2	42.8	43.3	6500	6120	6210	6280
B-60-100	41.7	43.0	38.8	41.2	6050	6240	5630	5970

Table C.4. Test Results for Modulus of Elasticity at 14 Days

Mix ID	SI Units (10^3 MPa)				US Customary Units (10^6 psi)			
	Test 1	**Test 2**	**Test 3**	**Avg.**	**Test 1**	**Test 2**	**Test 3**	**Avg.**
NCDOT	26.8	24.1	---	25.4	3.88	3.49	---	3.69
NCSU Mix 1	28.7	29.8	31.3	29.9	4.16	4.32	4.54	4.34
B-60-0	32.9	23.1	27.0	27.6	4.77	3.35	3.91	4.01
B-60-25	24.8	27.1	26.7	26.2	3.59	3.93	3.87	3.80
B-60-50	23.2	22.8	22.8	22.9	3.37	3.30	3.30	3.32
B-60-75	21.0	19.1	18.5	19.6	3.05	2.77	2.69	2.84
B-60-100	16.4	15.7	17.9	16.6	2.38	2.27	2.59	2.41
B-100-0	28.1	25.2	26.8	26.7	4.07	3.65	3.88	3.87
B-100-25	23.0	23.0	24.3	23.4	3.34	3.34	3.52	3.40
B-100-50	23.4	23.1	23.9	23.5	3.40	3.35	3.47	3.41
B-100-75	23.8	24.3	23.9	24.0	3.45	3.53	3.47	3.48
B-100-100	---	---	---	---	---	---	---	---

Table C.5. Test Results for Modulus of Elasticity at 28 Days

Mix ID	SI Units (10^3 MPa)				US Customary Units (10^6 psi)			
	Test 1	**Test 2**	**Test 3**	**Avg.**	**Test 1**	**Test 2**	**Test 3**	**Avg.**
NCDOT	27.6	21.8	25.6	25.0	4.00	3.16	3.71	3.62
NCSU Mix 1	34.4	40.3	34.1	36.2	4.99	5.84	4.94	5.26
B-60-0	31.6	28.5	31.9	30.7	4.58	4.14	4.62	4.45
B-60-25	29.9	31.4	28.4	29.9	4.34	4.56	4.12	4.34
B-60-50	26.2	27.0	27.6	26.9	3.80	3.91	4.00	3.90
B-60-75	24.0	23.3	21.4	22.9	3.48	3.38	3.11	3.32
B-60-100	---	22.1	21.7	21.9	---	3.21	3.15	3.18
B-100-0	23.2	21.2	---	22.2	3.36	3.07	---	3.22
B-100-25	---	---	---	---	---	---	---	---
B-100-50	---	---	---	---	---	---	---	---
B-100-75	---	---	---	---	---	---	---	---
B-100-100	---	---	---	---	---	---	---	---

Table C.6. Test Results for Modulus of Elasticity at 9 Months

Mix ID	SI Units (10^3MPa)				US Customary Units (10^6 psi)			
	Test 1	**Test 2**	**Test 3**	**Avg.**	**Test 1**	**Test 2**	**Test 3**	**Avg.**
NCDOT		3.88	3.49	3.69	3.04	3.12	3.12	3.09
NCSU Mix 1	37.7	38.2	33.9	36.6	5.47	5.54	49.2	5.31
B-60-0	42.0	38.7	38.0	39.3	6.09	5.48	5.51	5.69
B-60-25	33.1	31.7	31.9	32.2	4.80	4.60	4.62	4.67
B-60-50	25.4	29.9	30.5	28.6	3.68	4.33	4.43	4.15
B-60-75	27.2	26.0	27.8	26.9	3.94	3.77	4.03	3.91
B-60-100	24.9	26.1	25.0	25.4	3.61	3.79	3.63	3.68

Table C.7. Test Results for Flexural Strength at 14 Days

Mix ID	SI Units (MPa)				US Customary Units (psi)			
	Test 1	**Test 2**	**Test 3**	**Avg.**	**Test 1**	**Test 2**	**Test 3**	**Avg.**
NCDOT	3.55	4.07	4.03	3.88	515	590	585	565
NCSU Mix 1	5.55	5.17	---	5.36	805	750	---	780
B-60-0	3.93	3.96	3.83	3.91	570	575	555	565
B-60-25	3.72	3.93	3.93	3.86	540	570	570	560
B-60-50	3.93	3.59	3.79	3.77	570	520	550	550
B-60-75	3.48	3.93	3.38	3.60	505	570	490	520
B-60-100	3.24	3.52	3.21	3.32	470	510	465	480
B-100-0	3.93	3.96	3.93	3.94	570	575	570	570
B-100-25	3.62	3.55	3.86	3.68	525	515	560	535
B-100-50	3.52	3.59	3.28	3.46	510	520	575	500
B-100-75	3.34	3.52	3.10	3.32	485	510	450	480
B-100-100	---	---	---	---	---	---	---	---

Table C.8. Test Results for Flexural Strength at 28 Days

Mix ID	SI Units (MPa)				US Customary Units (psi)			
	Test 1	**Test 2**	**Test 3**	**Avg.**	**Test 1**	**Test 2**	**Test 3**	**Avg.**
NCDOT	4.00	3.96	4.14	4.03	580	575	600	585
NCSU Mix 1	5.90	5.48	4.62	5.33	855	795	670	775
B-60-0	4.45	3.86	---	4.16	645	560	---	605
B-60-25	3.83	4.03	3.96	3.94	555	585	575	570
B-60-50	3.74	4.14	3.83	390	540	600	555	565
B-60-75	3.69	3.55	3.83	3.69	535	515	555	535
B-60-100	3.59	3.55	3.48	3.54	520	515	505	515
B-100-0	4.14	4.17	4.14	4.15	600	605	600	600
B-100-25	---	---	---	---	---	---	---	---
B-100-50	---	---	---	---	---	---	---	---
B-100-75	---	---	---	---	---	---	---	---
B-100-100	---	---	---	---	---	---	---	---

Table C.9. Test Results for Flexural Strength at 9 Months

Mix ID	SI Units (MPa)				US Customary Units (psi)			
	Test 1	**Test 2**	**Test 3**	**Avg.**	**Test 1**	**Test 2**	**Test 3**	**Avg.**
NCDOT	5.27	4.62	5.17	5.02	765	670	750	730
NCSU Mix 1	5.41	4.69	5.17	5.09	785	680	750	740
B-60-0	5.48	---	5.48	5.48	795	---	795	795
B-60-25	5.58	4.31	4.96	5.29	810	770	720	765
B-60-50	5.45	5.17	4.69	5.10	790	750	680	740
B-60-75	5.07	5.03	4.93	5.01	735	730	715	725
B-60-100	4.55	4.41	4.10	4.36	660	640	595	630

Table C.10. Test Results for Splitting Tensile Strength at 14 Days

Mix ID	SI Units (MPa)				US Customary Units (psi)			
	Test 1	Test 2	Test 3	Avg.	Test 1	Test 2	Test 3	Avg.
NCDOT	3.41	2.00	3.07	2.83	495	290	445	410
NCSU Mix 1	3.14	2.90	3.79	3.28	455	420	550	475
B-60-0	3.62	3.07	3.03	3.24	525	445	440	470
B-60-25	2.93	3.28	3.10	3.10	425	475	450	450
B-60-50	2.69	2.86	3.03	2.86	390	415	440	415
B-60-75	2.90	2.48	2.65	2.68	420	360	385	390
B-60-100	2.28	2.21	2.62	2.37	330	320	380	345
B-100-0	2.79	2.48	2.86	2.71	405	360	415	395
B-100-25	2.62	2.65	2.90	2.72	380	385	420	395
B-100-50	2.76	2.79	2.55	2.70	400	405	370	390
B-100-75	2.34	2.79	2.65	2.60	340	405	385	375
B-100-100	---	---	---	---	---	---	---	---

Table C.11. Test Results for Splitting Tensile Strength at 28 Days

Mix ID	SI Units (MPa)				US Customary Units (psi)			
	Test 1	Test 2	Test 3	Avg.	Test 1	Test 2	Test 3	Avg.
NCDOT	3.59	3.00	2.93	3.17	520	435	425	460
NCSU Mix 1	4.07	4.65	4.10	4.27	590	675	595	620
B-60-0	2.90	3.65	3.59	3.38	420	530	520	490
B-60-25	3.24	3.34	3.24	3.28	470	485	470	475
B-60-50	2.55	2.90	3.45	2.96	370	420	500	430
B-60-75	2.86	2.52	2.93	2.77	415	365	425	400
B-60-100	3.03	3.56	2.52	2.74	440	385	365	395
B-100-0	3.10	2.86	3.34	3.10	450	415	485	450
B-100-25	---	---	---	---	---	---	---	---
B-100-50	---	---	---	---	---	---	---	---
B-100-75	---	---	---	---	---	---	---	---
B-100-100	---	---	---	---	---	---	---	---

Table C.12. Test Results for Splitting Tensile Strength at 9 Months

Mix ID	SI Units (MPa)				US Customary Units (psi)			
	Test 1	Test 2	Test 3	Avg.	Test 1	Test 2	Test 3	Avg.
NCDOT	4.62	3.76	4.07	4.15	670	545	590	600
NCSU Mix 1	4.21	4.17	4.17	4.18	610	605	605	605
B-60-0	3.72	4.86	3.31	3.96	540	705	480	575
B-60-25	3.24	2.45	4.34	3.34	470	355	630	485
B-60-50	3.17	3.72	3.17	3.36	460	540	460	485
B-60-75	3.28	3.24	2.83	3.11	475	470	410	450
B-60-100	3.28	2.72	2.72	2.91	475	395	395	420